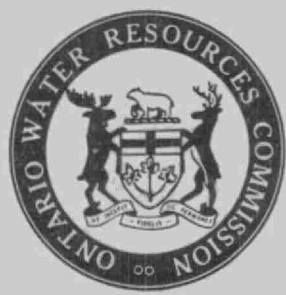


R. Hore



THE
ONTARIO WATER RESOURCES
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SENIOR COURSE

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WATER WORKS OPERATORS

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WATER WORKS OPERATORS

May 6th to 10th, 1968

ONTARIO WATER RESOURCES COMMISSION
SENIOR WATER WORKS OPERATORS' COURSE

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NEW DEVELOPMENTS IN WATER TREATMENT

A. B. Redekopp, P. Eng.

Supervisor, Water Works,
Division of Sanitary Engineering

Time does not permit in the three weeks of lectures and laboratory demonstrations to delve into the many new developments in water treatment to any great extent. In this lecture, we shall touch on some of the control refinements and processes which have been incorporated into newly-constructed plants and some of the studies which are being conducted at the present time. In broad terms, we will discuss new techniques in coagulation and flocculation, new concepts in water filtration and instrumentation.

PRE-TREATMENT

Polyelectrolytes

Much has been written lately on the subject of better controls of the coagulation-flocculation process and the use of polyelectrolytes, or polymers, as coagulant aids to assist alum or ferri-floc to produce a better and heavier floc.

These polyelectrolytes are long, linear chains of carbon atoms, colloidal in nature and action. Even in very dilute concentrations (ppb) these polyelectrolytes, or polymers, do a very efficient job of flocculation or coagulation. The long chain of carbon atoms (polymer) makes an excellent trap for colloidal impurities and turbidity particles in general. They are also thought of as making a "long-bridge" for coagulated particles to absorb on, and thus causing larger and heavier settling "flocs". Cationic polyelectrolytes are most frequently used in water treatment because of the positive charge which again, like alum, can neutralize the mainly negative charge of the impurities. It has been found for many raw waters that the amount of the prime coagulant (e.g. alum) can be cut down considerably when a small amount of a cationic polyelectrolyte is used.

Don Williams of Brantford has examined at least 25 commercially available polyelectrolytes as possible coagulant aids for the Grand River water. None were found to be sufficiently useful when compared to activated silica. Don suggests that the only means of determining whether any of the polyelectrolytes, or activated silica, is of any value for the raw water in question, would be to run jar tests. Jar tests are also very useful in deciding whether the coagulant aid should be added after the alum, before the alum, or coincident with the alum.

Filtrability Index Test

New techniques are being developed to help the operator decide on the optimum dosages of the prime coagulant and coagulant aid to use. AWWA Task Group 2720P is working on the development of a Filtrability Index Test. The Chairman of this Task Group reported at the 1967 AWWA Annual Conference that some progress has been made, but complete agreement on a simple test has not been obtained.

Zeta Potential

Zeta Potential (ZP) measurements can be made using a Zeta Meter. The ZP represents the magnitude of the negative charge present on the colloidal dirt particles present in the raw water. Normally, the ZP of surface waters varies in the range of -15 to -25 mv (milli-volts). Polluted waters may have much higher charges (up to -60 mv). The addition of alum (Al^{+++}) and cationic polyelectrolytes neutralizes the negative charge and reduces the Zeta Potential. Theoretically, a ZP of zero would indicate ideal conditions for coagulation-flocculation to take place; however, in practice this does not necessarily follow, and the ideal ZP for the raw water in question has to be established from experience.

The City of Jacques Cartier, Quebec, has adopted this system of control. Dominique Lamoureux, Waterworks Superintendent, stated that "after several months of working with the Zeta-Meter during which hundreds of analyses were made an awareness has been reached that knowledge of colloid chemistry is vital to the effective use of the instrument. Due to the nature of the apparatus, and the skill required to operate it, it is considered inadvisable that its facilities should be made available to any but fully qualified personnel (preferably university graduates).

In such hands, however, its value to the water treatment field, and the subsequent economics in operating and capital investment, will more than offset the initial investment required".

Pilot Filter to Control Coagulation

Several equipment suppliers have developed a pilot filter system to monitor coagulation. After the addition of the prime coagulant at the treatment plant, water is drawn by a small laboratory pump through the pilot filter, a polyelectrolyte is added to the pilot filter influent to prevent breakthrough, and the pilot filter effluent turbidity is measured on a continuously recording turbidimeter. Under these conditions, high turbidity shows that coagulation is poor, whereas low effluent turbidity shows that coagulation is good. If the pilot filter measures high effluent turbidity, the operator merely increases the alum dosage until the pilot filter indicates the required level of turbidity. Although the pilot filter is considered a luxury by many, it does have some merit and could be used as an additional tool.

Measurement of Floc Strength

Floc strength, or floc toughness, is defined as resistance to fragmentation by shear forces, induced by hydraulic velocity gradients. Floc strength is dependent upon the repulsive and attractive forces existing in raw water. Logically, then, if we can control these factors we should be able to control floc strength. These forces are electrostatic repulsion and Van der Waals attractive forces. Electrostatic repulsion forces are caused by the negative charged turbidity particles in the water. These charges are reduced by the addition of a coagulant such as alum (Al^{+++}). Van der Waals attractive forces have been said to be the major forces in attaching flexible polymer chains to colloidal particles including turbidity to permit bridging and growth of particles.

G.G. Robeck, U.S. Dept. of the Interior, who has been working on this subject of floc strength for some time now, has concluded that it was not possible to establish a numerical value for floc strength. The apparent floc strength was shown to be affected by changes in pH, coagulant and coagulant aid dosages, suspended solids concentrations, polyphosphate levels and temperature. Robeck reached the following general conclusions:

1. The highest rate of flocculation does not necessarily produce a floc most resistant to shear forces.
2. Alum floc alone is relatively weak and is apparently strengthened by incorporating solids into floc.
3. Sodium tripolyphosphate, if present when alum is added, may completely inhibit flocculation, but only slowly disperses an already formed alum floc.
4. The coagulant aids have a most pronounced effect in forming large, strong floc, but limited dosages must be employed for this purpose, because floc strength must be tailored to fit the overall treatment process, including filtration which may be adversely affected by a very high floc strength.

FILTRATION

Diatomaceous Earth Filtration

Diatomaceous Earth (D-E) filtration was developed prior to World War II, and was used extensively by the Armed Forces to produce potable water. Because of the smaller space requirements and the significantly lower capital costs, as compared to a conventional rapid sand plant, D-E filtration plants have been used for industrial, swimming pool and municipal applications. D-E filters are capable of removing turbidity, a substantial portion of colour and radioactive particles, iron, manganese, iron and sulphate-splitting organisms and a high percentage of coliform bacteria present in raw or pre-treated water.

Diatomaceous earth is composed of tiny skeletons of microscopic algae called diatoms (approx. 90% Si O₂ - S.G. = 2.1). D-E is basically a very fine, chemically inert powder, with a high porosity and permeability. Approx. 93% of its apparent volume is air space and it can absorb up to three times its own weight in liquid and still resemble a powder.

The filter septum, or filter cloth, on which the thin layer of filter media (D-E) is deposited and the filter shell, or housing, in which the septa are mounted, are the principal components of the D-E filter. Because of its irregularity, D-E is able to cling to the filter septum and build a precoat layer that completely covers the openings in the septum. This precoat is so permeable that liquid will pass through with a

minimum amount of pressure drop. The cake is virtually incompressible and forms some 2.5 million channels per square inch. These channels permit the free passage of liquids while screening out the suspended solids and semi-colloidal particles as small as one-tenth of a micron.

At this point, it would be well to compare the operation of the rapid sand filter with the D-E filter. The precoat layer of D-E in the D-E filter can be compared with the 24-30 inches of sand on the rapid sand filter. The filter septum of the D-E filter can be compared with the gravel and underdrain system of the rapid sand filter. At the conclusion of the lecture, we shall see some slides of the two municipal D-E filter installations in Ontario, viz: Marmora and Southampton.

Before filtration can begin (recommended filtration rate should not exceed 1 U.S. gpm/sq.ft.) the filter septa must be precoat with a thin layer of filter aid (D-E) which will form the actual filtering medium itself and will protect the septa from being clogged with slimes and turbidity. Filter aids settle out at a rate of about 4.5 ft. per min. If the internal flow velocity of the filter does not exceed this rate, then settling will occur. A precoat of 10 lbs. of filter aid for every 100 sq. ft. of filter area should be applied. The precoat slurry volume added to the filter influent should not exceed 25% of the circulation system volume.

Precoat filter aid and treated water are added to the precoat tank and agitated to keep the slurry in suspension. The precoat mixture is then circulated through the filter and back to the precoat tank using the recirculation pump. As the slurry passes through the openings in the filter septum (openings in the septum are usually no larger than 60-mesh screen size), the D-E filter aid will cling to the septum even though it is much smaller in size than the openings, and will very quickly bridge over all the openings to form a layer one-sixteenth of an inch thick. When the septa are completely coated, all the precoat is filtered out of the slurry and the water returning to the precoat tank is clear. This operation should take between 5 and 10 minutes. Once the septa are properly precoat, the filter can be put "on stream".

During the filter cycle, the turbidity in the incoming raw water is trapped on the surface of the precoat layer. To avoid blinding the tiny flow channels in the precoat by fine or slimy turbidity, it is necessary to inject additional filter aid

(D-E) into the incoming water. This is known as body feed which maintains the porosity of the filter cake by providing fresh flow channels. This helps to keep the differential pressure across the filter at a minimum and reduces the tendency of the filter to play. The ratio of body feed applied to turbidity is normally about 3 to 1. If the turbidity is slimy, a higher ratio is needed. The optimum rate of body feed is established by experience. The body feed septum should be capable of injecting at least 50 ppm filter aid into the incoming water stream. The slurry concentration of the body feed should not exceed 5% by weight.

If the operation of the filter throughout the cycle will not be continuous, some procedure has to be provided to keep the filter cake on the septum. This problem can be solved by providing reduced flow through the filter cake by means of a holding or recirculation pump. The holding pump should start automatically when the filter is stopped and provide a minimum flow of 0.1 gpm/sq.ft.

As the filtration progresses, the pressure differential eventually builds up to a point where the flow rate is very low. This occurs at about a pressure drop of 30 to 40 p.s.i. across the filter of a pressure D-E filter. Vacuum D-E filter usually operates until a headloss of 18 inches of mercury vacuum is reached.

The filter is then isolated from the rest of the system and drained. An efficient backwash system has to be provided that can consistently remove 95% or more of the filter cake after each cycle. The use of air-bump, reverse-flow, jetting and air-gurgle backwash, or a combination of these, is required for effective cleaning of pressure filters. For vacuum filters in which the septa are available for maintenance, the use of high velocity sluicing away of filter cake with treated water will ordinarily be satisfactory. Approximately 1/2% of the total water filtered is normally required to wash a filter effectively.

Pressure D-E filtration is accomplished in a pressure vessel similar to the pressure rapid sand filter, while vacuum D-E filtration is performed in an open tank where the filter septa can be seen at all times similar to a gravity rapid sand filter. The choice between a pressure or a vacuum filter installation depends to a great extent upon the particular

application. Whereas vacuum filters are operated to a low pressure differential of 6-8 p.s.i., pressure filters are normally operated to a pressure differential of 30-40 p.s.i. The higher available pressure differential may result in longer cycles and more efficient equipment use due to less frequent down-time for cake removal and precoating. The U.S. Army reported a study in the August, 1964 AWWA Journal evaluating D-E filters. The conclusion reached is as follows: "Vacuum D-E filters seemed to have many advantages over pressure D-E filters from the standpoint of simplicity of operation and maintenance".

Backwash water carrying a heavy load of used filter aid should not be discharged to the sanitary or storm sewer system. A separate lagoon or filter should be provided for this material.

For a detailed analysis of "Diatomite Filters for Municipal Use" refer to the 2710P Task Group report, published in the February 1965 issue of the AWWA Journal.

High-Rate Filtration

A good deal has been said and written about High-Rate filtration. Claims have been made that capacities of existing plants may be doubled or tripled with the minimum of expense, and that sedimentation or settling tanks have become obsolete and an unnecessary expense. When the raw water quality is good (low turbidity, colour, coliform levels, and algae concentration of below 1,000 A.S.U. per ml.) it is possible to consider higher filtration rates of up to 5 U.S. gpm/sq.ft. and elimination or reduction in the settling tank requirements, provided that a properly designed dual-media or multi-media bed is used and instrumentation is provided to control the treatment process.

High-Rate filtration is not altogether new. In 1935, Baylis of Chicago experimented with dual-media filters (anthracite coal (anthrafilt) over sand). He found that this roughing filter (the anthrafilt layer) greatly increased his filter runs. He reported in 1939, that a number of plants had duplicated his experiments and were using dual-media beds to great advantage.

Very little was written about this until the work of Conley and Pitman was published in 1960, on research work done at the U.S. Atomic Energy Plant, Hanford, Washington. They found that by controlling and monitoring the coagulation process they could obtain water of very low turbidity. To their amazement they found that this could be done at higher filtration rates. This was christened the Pit-Con process. Walter Conley subsequently joined the Neptune Micro-Floc Corporation and developed what has been termed the Micro-Floc process.

There are, of course, other firms in the business marketing similar processes. The difference lies mainly in the design of the bed (Neptune Micro-Floc Corp. used a multi-media bed consisting of three or more grades of filter media) and in the methods of controlling coagulation, which lies at the heart of the process. Previously, we discussed two methods of controlling the coagulation process, namely with the Zeta Meter and the pilot filter as well as the jar testing equipment. In addition, it is essential to provide continuously recording turbidimeters for both the raw and the filtered water.

The conventional rapid sand filter uses one grade of sand (0.45 - 0.55 mm size of approx. S.G. of 2.65) approx. 30 inches thick underlaid by graded layers of gravel as supporting media. Normally under these conditions the actual entrapment of suspended matter is restricted to the top two inches of the sand-bed. The remaining 28 inches of sand act as insurance against a serious turbidity breakthrough.

It can be visualized that the storing capacity for suspended matter of such a filter is considerably less than in a dual-media filter where the top 18 inches of the sand-bed have been replaced with a coarser and lighter anthrafil (0.8 - 1.2 mm. size of approx. S.G. of 1.75). Under ideal conditions, the entire 18-inch depth of anthrafil, along with 1-2 inches of sand, is available for the storage of suspended matter. This means that the headloss, instead of being concentrated in the top 2 inches in the conventional sand-bed, is distributed through a depth of 18-20 inches in the dual-media bed. This approach makes it possible to use higher filter rates with prolonged filter runs. With this process, however, it has been normal practice to apply a small dosage of a polyelectrolyte just prior to the water reaching the filter. This, of course, is in addition to the prime coagulant (alum), which is applied in its customary position.

It can be postulated that since the dual-media bed has a good deal of storage capacity in the anthrafilt layer, it should be possible to eliminate the sedimentation or settling tank and use the anthrafilt layer for this purpose. If the turbidity in the raw water is relatively low, say less than 15 ppm on the average, and maximum turbidity levels do not exceed 50 ppm, it should be possible to accomplish this, provided the water does not have any taste or odour problems and the algae concentrations are relatively low. In order to determine the design criteria for any particular installation, it is best to conduct pilot plant studies over a good period of time (preferably a minimum period of one year). In existing plants, where management wishes to consider the increase of filtration rates of existing filters, it has been common practice, in Ontario, to install a bank of pilot filters using various filter bed cross-sections taking water before and after the settling tanks to determine the required design criteria.

AUTOMATIC GRAVITY FILTERS

In recent years an automatic filter has been developed employing all the principles of standard filter design, but incorporating into the unit the backwash water. Two units described below differ only in the method of initiating the backwash cycle.

The backwash in the one unit is activated by a siphon arrangement and the other by a backwash control valve. These units may be constructed in one or more compartments.

Influent water is distributed uniformly over the surface of the filter bed, and percolates down to the false bottom where it is collected by strainers. Filtered water then proceeds from the underdrain compartment up to the backwash compartment, which is located over the filters, and out to service. In the backwash storage compartment the water level remains fixed, but water level in the control pipe rises slowly as filtering progresses and the bed collects dirt. This continues until the level differential reaches a predetermined point, representing a head loss of usually 5 feet.

At this point, the high level control opens the control valve, beginning backwash. As the water from the backwash compartment flows up through the filter bed, expanding and backwashing the sand discharging to waste, the level in the storage compartment drops. As this process continues, the water level in the backwash falls until the low level control closes the control valve completing the backwash cycle.

Rinse water then begins filtering through the bed, passing from the collection chamber up to the backwash storage compartment, where it remains until required for backwash. The filling continues until the previously fixed level is achieved and water again passes out to service.

These units have an application for industrial, municipal and private water supply systems. Temporary treatment requirements can be met during a period of development and construction. Such was the situation for the installation of a Graver Monovalve Filter in Pickering Township. This unit was installed and operated for a period of time while the standard filtration plant was being constructed.

Typical operating conditions:

Flow Rate:	2-3 gpm per square foot
Length of Service Runs:	1 to 2 days depending on influent turbidity and operating rate.
Head Loss:	Predetermined up to 5 ft.
Backwash:	15 gpm per square foot average rate.

AIR WASH FILTER

The air wash filter is used extensively in Europe, but to my knowledge has not been used in Ontario. It is worthwhile at this time to examine this principle.

Washing with Water and Air Scour

In this system a return flow of water always takes place from below upwards through the mass of sand at a rate insufficient to make the latter expand. At the same time there is injection

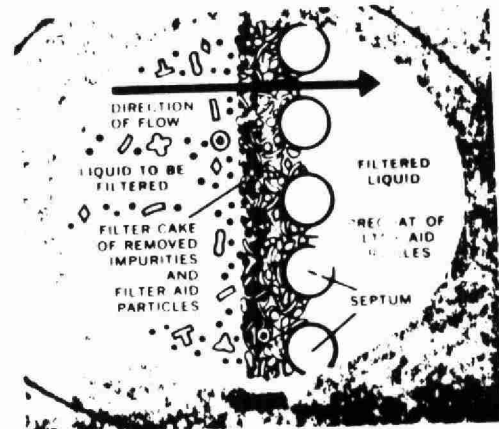
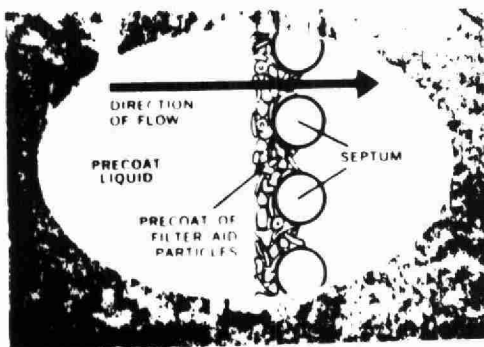
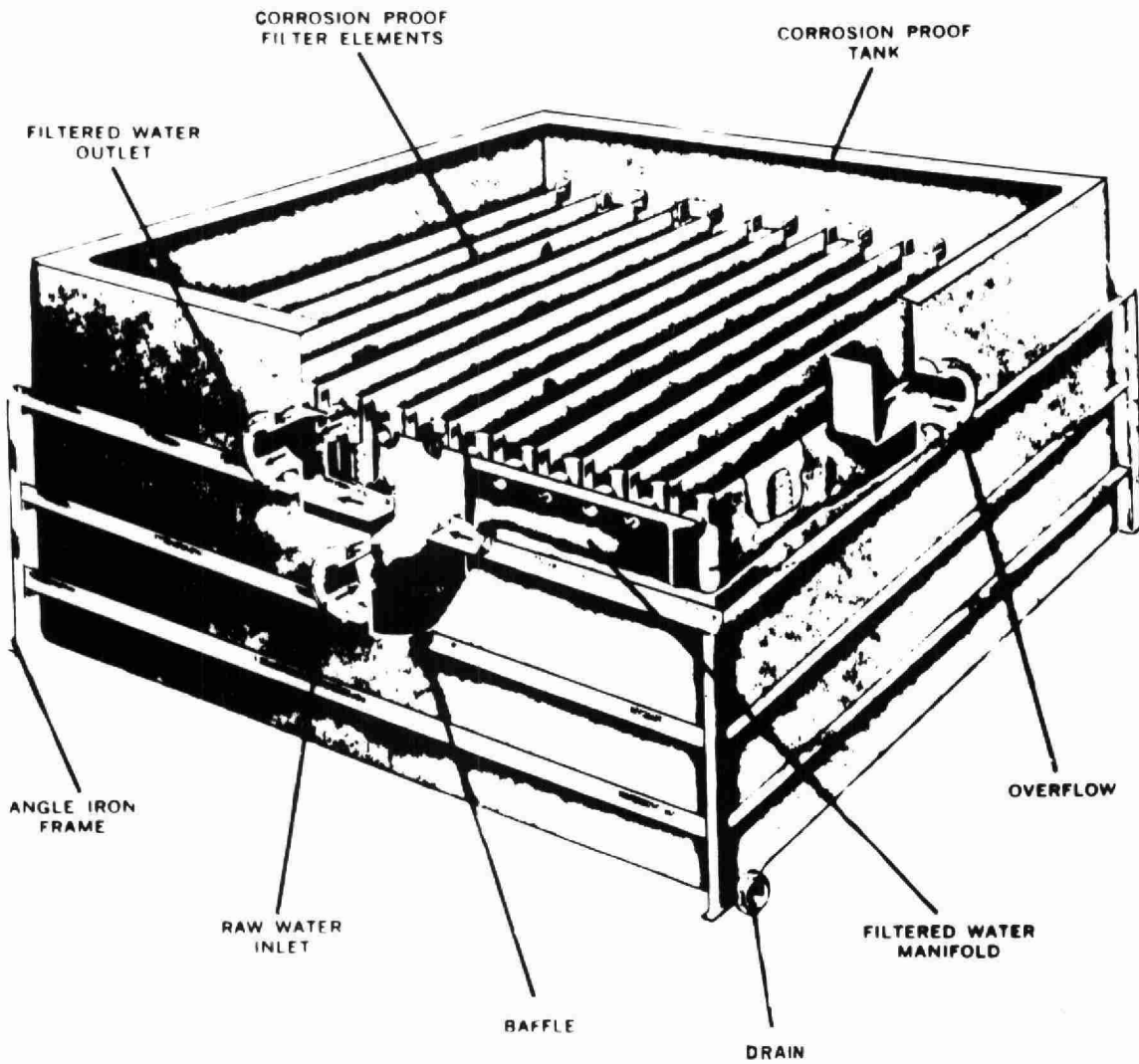
of uniformly-distributed air which rises through the sand and produces considerable turbulence in it.

If washing effected in this way is closely examined, it will be found that the sand is not stirred up in the true sense of the word. The sand mass remains more or less in place, but is subjected to vibration caused by the passage of a large number of air bubbles and this vibration detaches the impurities most efficiently and carries them off with the wash-water. As there are no eddies in the sand mass, the impurities can never descend to the lower part to form the famous "mud balls".

Further, the gravel support layers are not only unnecessary but should preferably be dispensed with or reduced to a small thickness of coarse gravel to prevent any of it becoming mixed with the filter mass by the energetic action of the air injected.

This particular washing system enables a larger grain size to be used. It eliminates the need for surface washing and sand expansion indicators. The washwater rate can be decided on and it is unnecessary to keep any special watch during washing operations; this facilitates automatic operation.

VERTICAL SCREEN D.E. VACUUM FILTER



AUTOMATIC VALVELESS GRAVITY FILTER

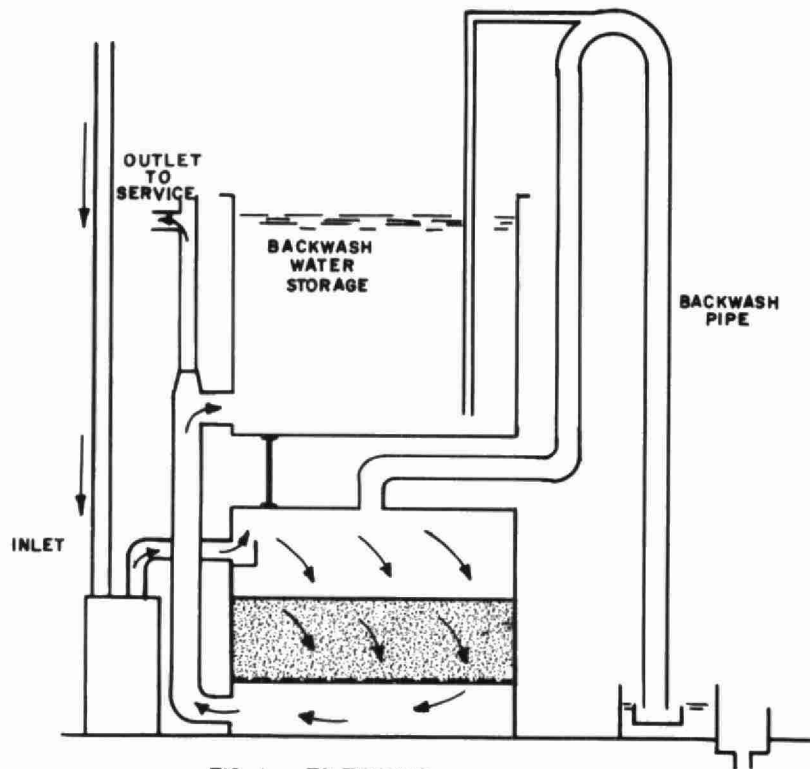


FIG. 1 FILTERING

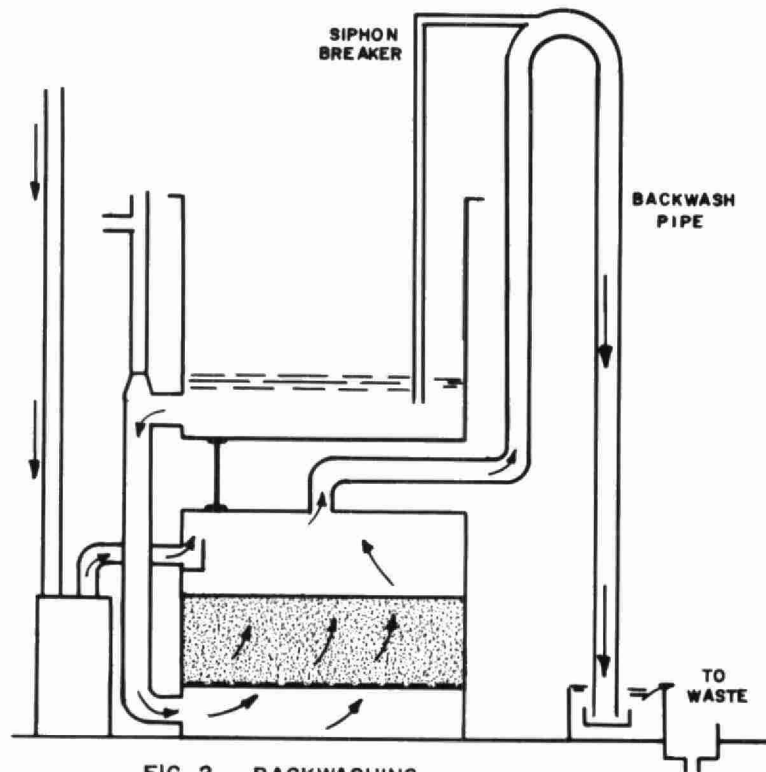
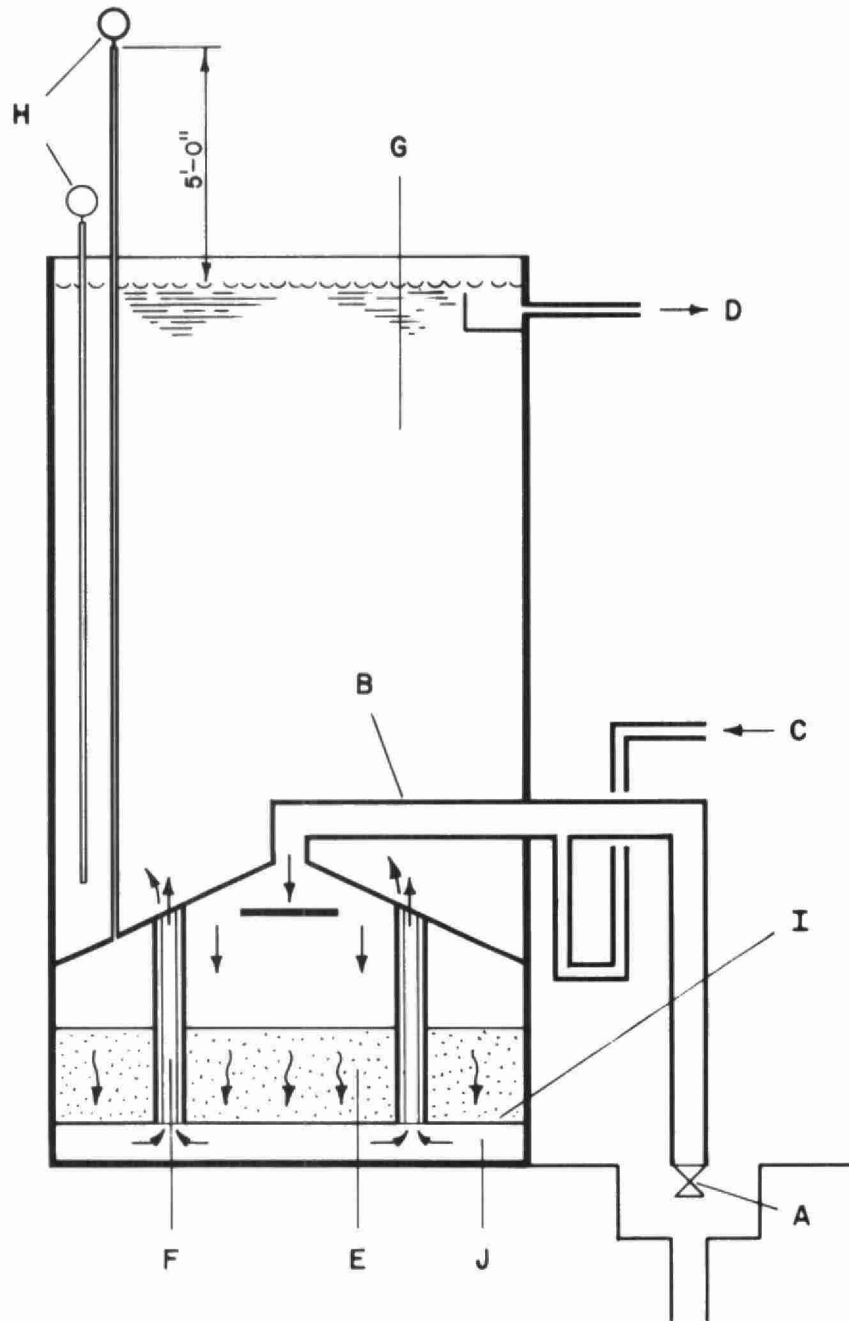


FIG. 2 BACKWASHING

MONO - VALVE GRAVITY FILTERTHIS DIAGRAMMATIC SKETCH SHOWS:

- A — BACKWASH CONTROL VALVE
- B — INTERNAL PIPE
- C — INLET
- D — OUTLET
- E — SAND BED
- F — CONNECTING DUCTS
- G — BACKWASH WATER STORAGE
- H — HIGH AND LOW LEVEL CONTROL SWITCHES
- I — STRAINER PLATE
- J — FILTERED WATER

OPERATION - MAINTENANCE OF
GAS CHLORINATORS

by P. D. Foley
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It would be best to begin this lecture by reviewing briefly the types of gas chlorinators as discussed in the Intermediate Course. There are several different types of gas chlorinators in common use today, all of which, however, operate on basically the same principal; namely the use of a water operated injector to create a vacuum in the machine, and hence draw a controlled amount of chlorine into the injector water supply.

Another class of gas chlorinator does exist, known as the pressure type. The pressure type uses the pressure available in the chlorine gas cylinder to add the chlorine at the point of application through a porous diffuser. The sole purpose of the pressure type chlorinator is to allow regulation of this flow and to provide the necessary safeguards. The pressure type unit is used mainly in the treatment of sewage effluent and in water treatment on those rare occasions when no power is available. Figure 1 shows a typical unit and consists of a chlorine pressure controller which provides a constant but adjustable chlorine pressure to a metering orifice. The flow of chlorine will be proportional to the square root of the pressure drop across the orifice. In cases where the unit discharges to atmosphere, a back pressure valve is required to provide a constant downstream pressure, against which the chlorine pressure controller can operate.

While the water diaphragm and bell jar type of vacuum chlorinator has not been widely manufactured for some years, This type of machine is still vary common in older plants. Figure 2 shows a flow diagram of the bell jar type unit and, although there are differences in the features of this group of units, most are similar in their basic operating principals to the model depicted.

As previously mentioned, the injector creates a suction pressure or vacuum in the machine and the function of all vacuum type chlorinators is to control the amount of vacuum and allow regulation of the rate of chlorine feed as desired. In the water diaphragm and bell jar unit the injector is allowed to pull a mixture of water and gas into the unit and the feed rate is regulated by varying the ratio of these two materials in the meter vacuum control unit as shown in Figure 2. The mixture flowing out of the

unit is constant, the water flow into the meter vacuum control unit is varied by the position of the submergence tube and the difference in flow controls the amount of vacuum on the metering orifice in the bell jar. The amount of negative chlorine pressure in the bell jar is constant and controlled by the chlorine pressure reducing valve. Therefore, the chlorine pressure reducing valve produces a constant negative pressure on one side of the chlorine metering orifice, and the submergence tube controls the constant, but adjustable negative pressure on the other side of the chlorine metering orifice. The result is a constant controlled flow of chlorine,

When a chlorinator stops functioning, the cause is usually loss of vacuum for one reason or another. This may be due to restriction of the injector orifice by dirt or other foreign material, or it may be due to a vacuum leak in some component or joint of the machine. Since the older type of chlorinator such as this has an auxiliary injector water supply, the chlorine becomes moist before entering the injector and corrosion problems are quite common, particularly in view of the fact that the materials used in their manufacture are less corrosion resistant than the plastics which are now commonly used in chlorination equipment. Rubber gaskets, therefore, should be frequently replaced as deterioration of these can allow a vacuum leak and hence the machine breaks down. Where iron is present in the auxiliary water supply, the various sight glasses in these units can become coated quite quickly and frequent maintenance, therefore, should be carried out to ensure that these are kept clean. In this way any malfunction of the unit can be quickly traced and the necessary steps taken to correct the fault. Where the auxiliary water supply drops below 40° in winter, icing in the bell jar is quite common although this seldom interferes with operation of the chlorinator. Icing can be minimized by installing a heat lamp above the bell jar although a better cure is to install an in-line heater on the auxiliary water supply which will raise the temperature sufficiently to eliminate the icing. So far as calibration of this type of chlorinator is concerned this can be checked quite readily by relating the chlorinator setting to the loss of weight indicated by the scale on which the chlorine cylinders are mounted. Any discrepancy between these figures should be investigated and the cause determined. Since all machines of this type basically use a metering orifice and a factory calibrated scale, the usual cause of an improper feed rate is dirt on the metering orifice. The zero level on the scale however, should be checked from time to time to make sure that this is at the proper position.

One extremely important part of any gas chlorinator system is the vacuum relief line. This should always run to the outside of the building and should contain no traps where internal condensation might collect. In the case of the bell jar chlorinator the

vacuum relief line is particularly important as blockage of the line may lead to flooding of the chlorine cylinder. When the line is blocked, the water level in the bell jar can then rise above the end of the chlorine inlet tube. Incidentally, a chlorine cylinder, which has had water in it, is dangerous as internal corrosion can quickly take place.

Other maintenance which should be carried out consists of such normal precautions as cleaning out water line strainers and chlorine gas strainers if supplied, and checking injector and auxiliary operating water supply pressures from time to time.

A second type of chlorinator which is no longer produced but is still in use is known as the mechanical diaphragm unit, with water seal. Most of the remarks which apply to the bell jar unit also apply to the mechanical diaphragm machine, with the exception of such obvious differences as the lack of bell jar icing with this machine. The remarks with respect to the vacuum relief line are perhaps even more important so far as the mechanical diaphragm unit is concerned, since a plugged vacuum relief line results in a flooded diaphragm unit. This has usually to be returned to the factory for overhaul which may be a somewhat costly repair.

The machine which is now almost universally employed for new chlorine systems or for replacement of old systems is the plastic diaphragm machine. A flow diaphragm of this unit is shown in Figure 3, and as you will note, this is an extremely simple machine compared to the old units. Basically however, the new machines employ the same principles as the old units and derive their operation from the vacuum created in a water jet operated injector. The vacuums used nowadays are, however, much greater than with previous machines and the vacuum may be as high as 80 inches of water, compared to 10 to 12 inches which used to be used. Among other advantages this higher value allows a much wider operational range and the flow is now indicated on a rotameter which has an even calibration as opposed to the water manometer which has a sq. rt. scale giving extremely close markings at the bottom end of the range. One basic difference in a modern machine is that dosage control is achieved by varying the orifice size (unit shown illustrates a V-notch variable orifice) and keeping the vacuum constant. Supplementary controls such as flow proportioning devices can be added to give a secondary form of control by varying the vacuum.

A chlorine gas meter should be chosen so that the anticipated feed rate is somewhere in the middle third of the range. In this way good accuracy and readability will be gained while a sufficient margin for increased dosage will be available. In many cases it would probably be advisable to have two chlorine gas meters supplied with the machine as seasonal differences can make quite

large changes in the required chlorine dosage. With most chlorinators a wide selection of injector throat sizes is available and the manufacturer should be supplied with sufficient information to size the injector for most efficient operation. An item which is frequently omitted from a chlorinator installation is the provision of a water pressure reducing valve on the injector water supply. If the chlorine solution is being injected against a negligible or non-existent pressure such as in a open well application, the supply pressure required by the machine can often be cut down to quite a low value which will still pull the full capacity of the machine. All too frequently a machine is installed with an injector water pressure of 80 or 90 psi and this results in needless waste of water. If the back pressure is no more than 5 psi, most machines can be made to operate quite adequately at 30 to 35 psi and the water consumption can often be halved. As an example of the amount of water used a 50 lb per day chlorinator will use 4 gallons per minute at 25 psi, while at 80 psi this consumption will be around 7 gallons per minute is possible, or 4,200 gallons per day, if the water costs 25¢ per 1,000 gallons, this could amount to something of the order of \$350 per year, to say nothing of the unnecessary noise and wear which the higher pressure causes. Where the point of application is by necessity, in a line under pressure of course, the manufacturer will usually specify the injector requirements, when the line pressure is known and a suitable booster pump must be installed to raise the injector water operating supply pressure. Since the requirements are normally high pressure and low volume, a turbine type booster pump is usually used. Turbine pumps are particularly subject to wear which will lower output pressure and volume. Always oversize the pump by at least 15 percent with respect to pressure and volume and ensure that the water supply is free of abrasive materials.

Maintenance on modern chlorinators is usually confined to checking chlorine connections replacing O-Rings, and cleaning the injector water strainer and the chlorine gas strainer. The absence of moist chlorine in the chlorinator metering system plus much more resistant materials of construction means that deterioration due to corrosion has been minimized. The replacement of rubber by flexible plastic for O-Rings and gaskets had added months to their service life. An annual replacement program, however, is still the best preventative maintenance. One extremely important precaution must be taken with these machines and that is the elimination of the possibility of chlorine in a liquid form entering the unit. As you are no doubt aware, although the chlorine is taken from the cylinder in the form of gas, the compressed chlorine is a liquid in the cylinder and it is possible to get reliquifaction of the chlorine at the chlorinator if the chlorine cylinder temperature is significantly higher than the temperature in the vicinity of the chlorinator. I will not describe the physics of this phenomenon but the effect of the liquid chlorine on the plastic components is quite serious and

leads to deterioration over a period of time. In an extreme case the whole interior of the chlorinator can freeze solid if a large amount of liquid chlorine enters the machine. The guiding principle, therefore, in any installation is to have the chlorine cylinders at the same temperature as the chlorinator or, if possible, at a lower temperature.

Before concluding I would like to make a few remarks with respect to several of the accessories often supplied with a chlorinator. Distributing panels are frequently supplied to allow simultaneous application at two or more different points in the treatment system. These panels must only be used for proportioning the amount of chlorine which the chlorinator is set to feed and cannot be used to cut down the total chlorine feed rate. If the dosage to one of the application points must be reduced for instance, a new set of conditions must be worked out which involve adjustment of the total feed rate at the chlorinator. If, for instance, the chlorinator is set at 60 lb and the rotameters are adjusted to give a two to one ratio to the two points of application, this means that 40 lb will be given to one point while 20 lb will be given to the other point. If, however, it is desired to reduce the rate at the one point from 20 to 15 lb, then the chlorinator must be readjusted to give the new total which is 55 lb instead of 60 lb, and the new ratio should be set on the rotameters which would be 40 to 15, or 3 to 8. Incidentally, although valves are shown on the inlet and outlet of each rotameter, the inlet valves are solidly for shut-off purposes should the rotameter ever be damaged or removed for cleaning. All throttling should be done on the outlet valves, one of which should be fully open. It is not necessary to use more than one of the outlet valves to adjust the dosage ratio. This, of course, refers to a panel with two rotameters. However, no matter how many streams there are, one outlet valve should be open at all times and all the inlet valves should be fully open under normal operation.

Of equal importance in the operation of a satisfactory chlorination system is the method used to distribute the chlorine solution to the water. For small sized lines a plastic tube and corporation cock are usually used, and I am sure you are all familiar with them. I am also sure that you realize that the tube must be withdrawn to the limit of the chain, if you are to avoid cutting the end from the diffusertube. In larger installations a diffuser tube with holes on either one or both sides is used. If the tube is withdrawn, make sure it is reinstalled with the holes discharging across the line of flow.

In gas chlorination installations the current trend is to residual control using a residual cell and indicator or recorder to control the dosage of the chlorinator. In operation the residual

cell continuously samples the plant effluent and determines the chlorine residual. The result is automatically compared with the desired residual as set by a pointer on the unit by the operator. If the residual deviates more than an adjustable distance from the desired residual, an electrical signal is sent to the chlorinator dosage control to adjust it appropriately. The residual, therefore, is continuously monitored and adjusted.

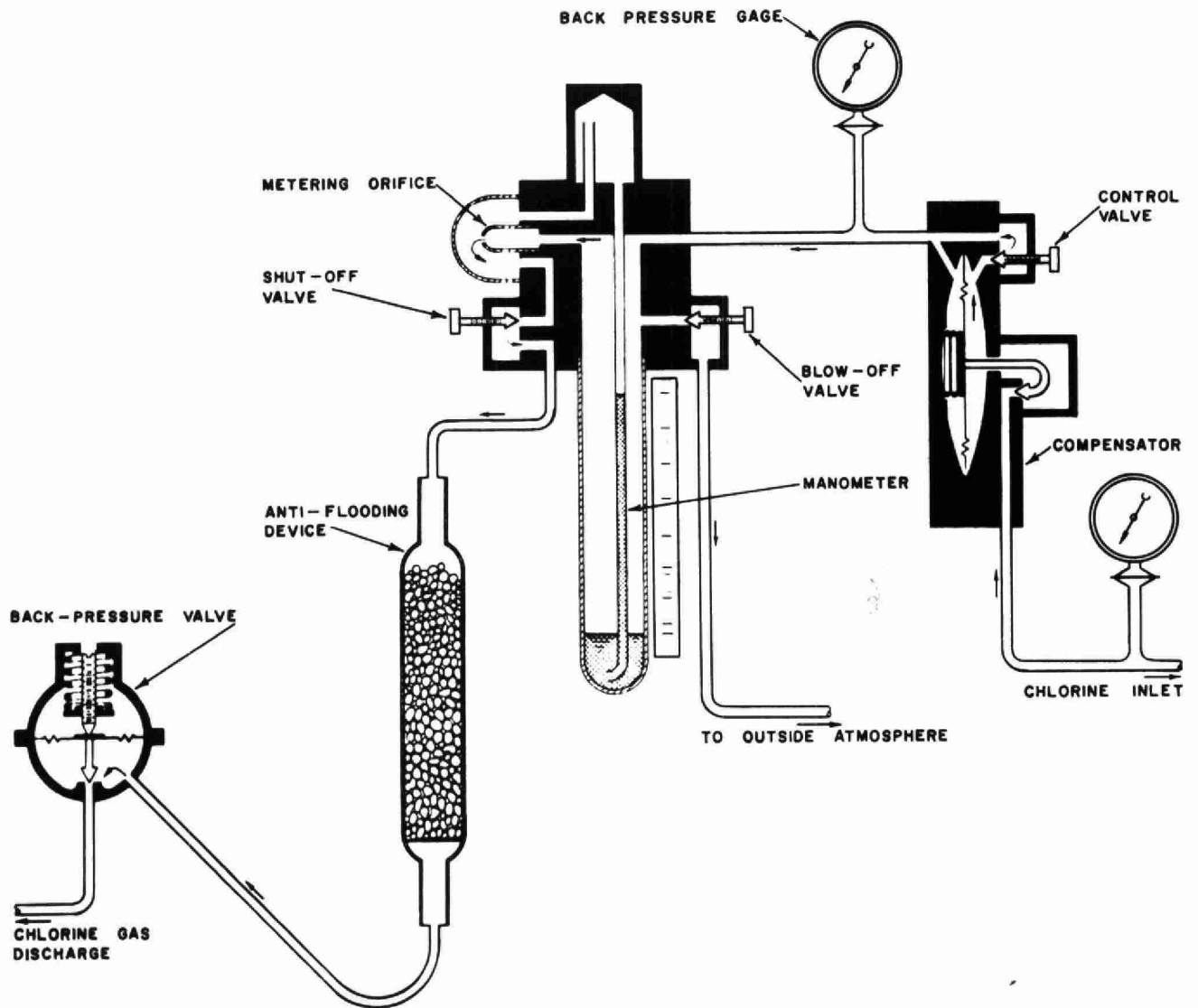


FIGURE I

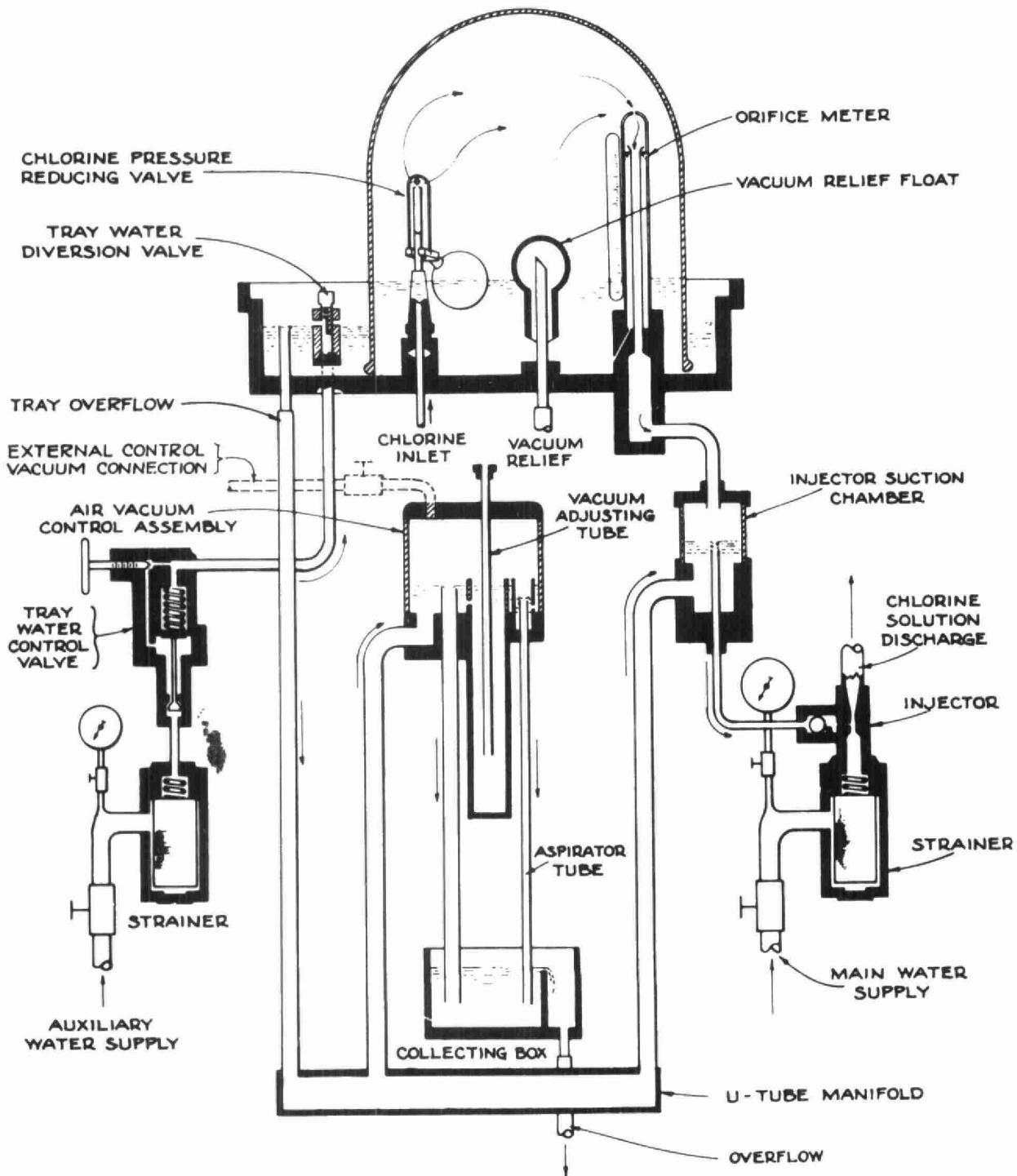


FIGURE 2

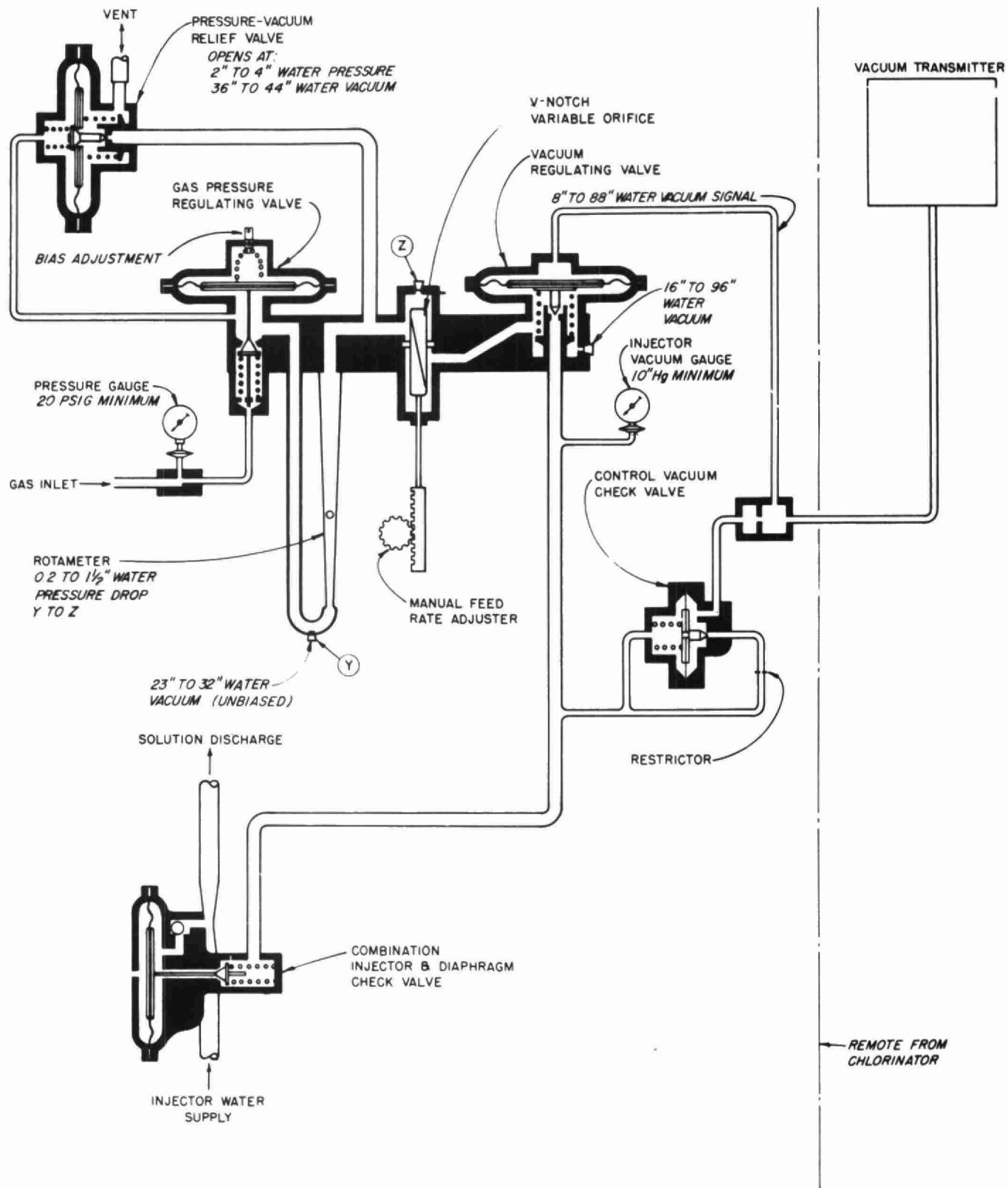


FIGURE 3

COAGULATION AND FLOCCULATION OF WATER - III

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REVIEW

In the first two lectures we were mainly concerned with the principles involved. We spoke of turbidity in the raw water appearing in the nature of a "fog" which could pass right through filters. We learned that the particles of turbidity were negatively charged. We learned that by means of the addition of certain chemicals, called coagulants, we could produce in the water billions of positively charged particles. These positively charged particles that we had added neutralized the negatively charged turbidity particles.

We learned that when this neutralization had taken place during coagulation, by means of continued stirring we could cause the original turbidity particles, plus the "floc" particles that we had added, to grow together and to become so large that, instead of the turbid water having the appearance of a "fog", it now resembled a "snowstorm".

We learned that changing the "fog" into a "snowstorm" was just what was needed, since most of the particles would now settle out and those that did not settle out would now be sufficiently large that they could be retained by filters.

We discussed the reason for settling basins. We went into a lengthy discussion of various types of turbidity and how they affected coagulation, and in turn how the dissolved salts in the water could affect coagulation regardless of the type of turbidity present. We described coagulants, coagulant aids, and various types of advanced instrumentation to determine results.

With all the new instrumentation coming into use in the government agencies and the very large water treatment plants, one practical fact stands out alone, and that is: the time honoured "Jar Test" still has, and will continue to have for many years to come, a most valuable place in water plant practice ranging from the largest plant to the smallest.

We mentioned the jar test in one of the previous lectures and we did give a demonstration of its operation. However, we consider the jar test so important that we intend to devote to it all of this lecture. A few remarks regarding equipment, its operation, and what can be learned from it are in order, but realize that this is mainly a lecture demonstration from which we hope to develop useful discussion from the floor.

FOR THE RECORD

A most complete description of the JAR TEST is to be found in: "Simplified Procedures for Water Examination", AWWA-M-12, Simplified Laboratory Manual, available from: American Water Works Association, 2 Park Avenue, New York, N.Y. 10016, U.S.A.

APPARATUS REQUIRED FOR JAR TEST

- 1) A six-place stirring machine of the type manufactured by Phipps and Bird Inc., Richmond, Virginia, available from all scientific supply houses in Canada. All other equipment mentioned herein is similarly obtained.
- 2) 1500 ml. Beakers, low form, pyrex -- at least 8, 6 for use, the remainder as spares.
- 3) A plastic pail, household type, capacity in excess of 2 gals. for collecting the sample. Rope attached to handle if sampling from a stream.
- 4) A 1000 ml. graduated cylinder.
- 5) Measuring Pipettes: The AWWA Manual M-12 calls for the use of 1, 5, and 10 ml. measuring pipettes (Mohr type) all graduated in 0.1 ml. steps. You will find great use for these pipettes when you have gained some experience and wish to branch out into some investigational work, particularly with coagulant aids. However, to start out, and particularly for making some first simple observations with one coagulant, such as aluminum sulphate, it is suggested that you obtain one 0.5 ml. and one 1.0 ml. Ostwald type pipettes. Ostwald pipettes are normally used in medical and biological work and have the advantage that you blow out the contents rather than waiting for them to drain.

After some experience you may wish to acquire a considerable array of various sizes and types of pipettes. Immediately after use, all pipettes should be thoroughly rinsed with tap water, but preferably distilled water if available.

DOSING SOLUTIONS AND SUSPENSIONS

There are a number of substances used in the coagulation of turbid and coloured waters. They may be classed as follows: true coagulants, coagulant aids and sources of alkalinity.

Common Coagulants

Aluminum Sulphate ($\text{Al}_2 (\text{SO}_4)_3 \cdot 14 \text{H}_2\text{O}$), also called alum, filter alum, and paper maker's alum.

Ferrous Sulphate ($\text{FeSO}_4 \cdot 7 \text{H}_2\text{O}$), also called copperas.

Ferric Sulphate ($\text{Fe}_2 (\text{SO}_4)_3$), also called ferrisul.

Ferric Chloride (FeCl_3 .)

Sodium Aluminate (NaAlO_2)

(any papers originating in the United Kingdom, dealing with water treatment, would likely mention "aluminoferric" -- this is none other than our old familiar alum containing a trace of iron which makes it not suitable for making white paper, but just as good for water treatment as our normal alum).

Coagulant Aids

Activated silica, synthetic organic materials known as polyelectrolytes, clays, kaolin, and stone dust.

In some waters, the addition of alkalinity is often required to produce the desired hydrous oxide floc, and in others it is necessary to adjust the pH upwards after the floc has started to form.

Suitable Sources of Alkalinity

Calcium Oxide (CaO), also called lime, quicklime, unslaked lime, burnt lime.

Calcium Hydrate (Ca(OH)_2), also called calcium hydroxide, hydrated lime and slaked lime.

--- both varieties of lime should be of the high calcium type.

Sodium Carbonate (Na_2CO_3), also called soda ash.

Sodium Hydroxide (NaOH), also called caustic soda, soda lye and lye.

In the above, calcium oxide and calcium hydrate are considered solely as sources of alkalinity, but it should be realized that in excess lime softening, which carries a very strong coagulation effect, both of these chemicals are used as true coagulants. In alkalinity adjustment, it has been necessary to reduce the alkalinity at times, and thus lower the pH, and in such cases sulphuric acid, H_2SO_4 , can be used, but its use is such a rarity that it is beyond the scope of these primary lectures.

Any solutions and suspensions to be used in this work should be made from stock materials actually used or offered for use in water plant practice, rather than using those very pure analytical grades found on the chemist's shelf.

Distilled water is the only water to be considered for making the required solutions and suspensions.

Two other items of equipment are required: a small pan balance, capable of weighing up to 100 grams, having an accuracy of 0.1 gram, and a one litre glass stopped, volumetric flask.

Stability of the Stock Solutions

Aluminum sulphate, clay, kaolin, stone dust, calcium hydrate, calcium oxide solutions or suspensions, if maintained well capped, will remain stable for months. Solutions of ferric sulphate, ferrous sulphate, ferric chloride, and sodium aluminate require making up fresh daily.

MAKING SOLUTIONS OR SUSPENSIONS

Alum

Since all of the solutions or suspensions are made in the same manner we will describe the making of only one, aluminum sulphate:

Carefully weigh out on the balance exactly 14.3 grams of aluminum sulphate ($\text{Al}_2(\text{SO}_4)_3 \cdot 14 \text{ H}_2\text{O}$) and by means of a folded slip of paper transfer all of this amount of the alum into the 1000 ml. volumetric flask. Add distilled water up to the measuring mark. Stopper the flask and then cause the aluminum sulphate to go into solution by inverting and/or swirling the flask.

After the aluminum sulphate crystals are in solution (there will be a small trace of inert material that will never completely dissolve), pour the contents of the flask into 16-oz. plastic capped bottles (avoid metal caps) and label "stock alum".

The concentration of this solution, and any other solutions based on 14.3 grams of the reagent made up to one litre, is such that when 1.0 ml. of the solution is added to a one litre sample of water in the stirring machine, we have produced in that sample a concentration of "one grain per Imperial Gallon".

In the U.S.A., since the U.S. Gallon is of a different size from the Imperial Gallon, it is necessary to start with 17.1 grams made up to one litre, and then 1.0 ml. of this solution when added to a one litre sample of water represents a concentration in the sample of "one grain per U.S. Gallon".

Those who wish to work exclusively in parts per million, it is necessary to weigh out exactly 10.0 grams of the reagent (alum), make it up to one litre, and then 0.1 ml. of this solution when added to a one litre sample of water represents a concentration of 1.0 part per million, whereas a full 1.0 ml. in a one litre sample represents 10. parts per million.

Coagulant Aids

There are so many proprietary coagulant aids, often made up to different concentrations, that we cannot deal with them here. The technical sales personnel offering these products will bring samples of them to your plant ready for use in the stirring machine, or they will give you the compound and tell you the concentration to which it should be made up.

The same is true for activated silica, but for the record, we are including a useful laboratory method of making up this valuable coagulant aid.

Activated Silica Solution for use in Jar Tests

(a) Stock Solution of Sodium Silicate

Weigh 348.4 grams of sodium silicate solution ("N" Brand, sodium silicate solution as made by Philadelphia Quartz Co., Philadelphia, Pa., U.S.A., and National Silicates Limited, Toronto, Ontario). Dissolve this sodium silicate solution in 500 ml. of distilled water and then dilute to one litre with distilled water. If capped in bottles, this solution will remain stable indefinitely.

(b) Stock Ammonium Sulphate Solution

Weigh 66.0 grams of ammonium sulphate ($\text{NH}_4)_2\text{SO}_4$). Dissolve in 980 ml. distilled water. Kept in capped bottles, this solution is stable indefinitely.

(c) Preparation of the Activated Silica Solution for use in the Jar Test

(1) Place 20 ml. distilled water in a 100 ml. graduated cylinder. Add 10. ml. of stock sodium silicate solution and invert four times to mix.

(2) Add 10. ml. stock ammonium sulphate solution and invert 10 times to mix. Let this mixture age for 5 minutes, exactly. Then dilute to 100 ml. with distilled water and invert 10 times to mix. The activated silica solution is now ready for use in the Jar Test. It should be made fresh daily.

(3) Each 0.1 ml. of this activated silica solution represents a dosage of 1.0 part per million when added to a one litre water sample in the Jar Test, and, therefore, 1.0 ml. of this activated silica solution represents 10. parts per million of activated silica when added to a one litre sample of water in the Jar Test.

OPERATING THE JAR TEST

- (1) Rinse six 1500 ml. beakers with tap water and let drain upside down for a few minutes. Beakers that have had several days of use should be scrubbed inside and out with a beaker brush and a household dishwashing detergent, finishing with a thorough rinse of tap water.
- (2) Clean the stirring machine paddles with a damp cloth.
- (3) Collect a sample of the raw water in the plastic pail and complete the following steps 4 through 8 within 20 minutes. Throw away the sample and start all over again if anything interrupts during this critical stage. Otherwise, the settling of high turbidity, loss of charges on particles, and an increase in sample temperature, due to room heat, may give erroneous results.
- (4) Stand beakers right side up and pour more than one litre of raw water into each one.
- (5) Taking one beaker at a time, pour some of the raw water back and forth between the beaker and the 1-litre graduated cylinder, to make sure that the turbidity has not had an opportunity to settle out. Finally, fill the graduated cylinder to the 1-litre mark and discard any excess water in the beaker. Pour the measured 1-litre sample into the beaker.
- (6) Place all six beakers containing the measured 1-litre samples on the stirring machine.
- (7) Lower the stirring paddles in such a manner that the paddles do not touch the sides of the beakers when stirring, but also in such a manner that they are offset as far as possible from absolute centre of the beaker. Start the stirring machine at a speed of 80 RPM.
- (8) Now the operator must add varying dosages of a coagulant to these beakers. Let us assume that we have a water on which we have not performed a jar test. What dosages shall we add? We have to start somewhere. Therefore, let us add stock aluminum sulphate to these beakers in this order: 0.5 ml. to the first beaker, 1.0 ml. to the second, and so on, in increasing 0.5 ml. steps until the last one is reached at a dosage of

3.0 ml. of alum solution. Expressed as grains per Imperial Gallon our dosage becomes:

Beaker No.	1	2	3	4	5	6
ml. solution per litre	0.5	1.0	1.5	2.0	2.5	3.0
Grains per Imp. Gal.	0.5	1.0	1.5	2.0	2.5	3.0

(The pipettes should be flushed out with the aluminum sulphate solution 2 or 3 times before use).

- (9) The addition of these dosages of alum to the six beakers will take about one minute. After the last beaker is dosed permit the stirring machine to operate at 80 RPM for one additional minute and then over the next 30 seconds reduce the stirring speed to 30 RPM and permit stirring at that speed to continue for exactly 15 minutes.
- (10) During the period of stirring at 30 RPM, make notes about the first appearance of floc in the various beakers and the type of floc found. Is it a pinpoint floc, is it rugged or soft and feathery of the type that could easily break up and pass through filters? Is the floc the same as you obtain in plant operation, or is it better in some respect?
- (11) At the end of the 15 minutes slow stirring stop the machine and permit the samples to settle for 5, 15, 30 or 60 minutes as the case may require.

Observe the floc and record the order of settling. Describe the results as poor, fair, good or excellent. A hazy sample indicates poor coagulation and can mean that the wrong coagulant is being used, or if it is the right coagulant, then not enough has been used.

Properly coagulated and flocculated water contains floc particles that are well formed, and particularly the water between the floc particles should be clear.

The lowest coagulant dosage which brings down all of the turbidity, leaving the water brilliantly clear, should be tried in plant operation, but remember this one point: the jar test machine is generally more efficient than operations in plant, therefore, do not be surprised if you require to use more coagulant in plant operation than found in the jar test.

EVALUATION OF JAR TEST RESULTS

Let us assume that this is your very first jar test, that you used dosages of aluminum sulphate ranging all the way from 0.5 grains per Imperial Gallon up to 3.0 grains per Imperial Gallon. (you could have gone from 2.0 grains up to as high as 5.0 grains and even higher, if indicated).

Let us assume that in this first jar test that the dosage of 2.0 grains per Imperial Gallon was the lowest dosage that gave you what you wanted: a fast forming, fair sized, rugged, and fast settling floc.

Now, proceed to try all the other coagulants in the same manner. One of them may give you a faster forming and settling floc at only 1.5 grains per Imperial Gallon, but the settled water at any dosage may not be completely clear -- this is obviously not the coagulant suited to your raw water.

Another coagulant may require a higher dosage than the 2.0 grains to give you fast forming, fast settling floc, but even though the final settled water is clear, there may be too much natural colour left in the water, and this coagulant may be more expensive -- again, this is obviously not the coagulant best suited to your raw water.

By keeping on in this manner for one full day, two days at the very most, you should be able to select the most suitable primary coagulant for your raw water. Once this selection has been made you will generally never require to do it again, because even though rivers and lakes change in amount of turbidity present, they generally do not change on the basis of the type of coagulant best suited to them.

Let us assume that you have found aluminum sulphate (it could have easily been one of the others) to be the coagulant best suited to your raw water and at the jar test indicated dosage you obtain a good floc in plant operation; the water entering the filters is low in turbidity; you have long filter runs; you have no trouble with algae plugging filters; the effluent turbidity from filters is suitably clear, and when using activated carbon you have no trouble with carbon passing through filters to give black water complaints. If you find all of these desirable conditions, you are virtually on easy street, and your investigational work on coagulation is essentially finished and all you need to do is to run jar tests at

stated intervals to select the correct dosage of aluminum sulphate to bring down the turbidity existing at the time in question.

There are some waters in which aluminum sulphate, unaided, will produce these desirable results, but there are a great many waters where aluminum sulphate (and the other coagulants, also) require considerable aid from coagulant aids.

Let us assume then that aluminum sulphate alone at a dosage of 2.0 grains per Imperial Gallons gives us the best possible results in both jar tests and plant operation, but in being the best it still leaves much to be desired in the nature of floc, speed of settling, clarity between particles, colour removal, does not give low turbidity on filters, poor filter runs, filters plug too easily with algae, and when applying carbon some passes through filters. If you find some or all of these conditions, then you can say that, although aluminum sulphate has been shown to be the best primary coagulant, it certainly requires some help.

What should one do?

SELECTION OF A COAGULANT AID

We have already found that a dosage of 2.0 grains per Imperial Gallon (it could have easily been some other dosage) of aluminum sulphate is the best that we can obtain.

So now we proceed to run a number of jar test series in which all the six beakers in the stirring machine are dosed with 2.0 grains per Imperial Gallon of aluminum sulphate.

However, before putting the aluminum sulphate into the beakers we should previously treat the water in the beakers with varying amounts of lime ranging from 0.5 grains per Imperial Gallon up to 3.0 grains per Imperial Gallon. In this way, we might well find that 1.5 grains of lime, plus the 2.0 grains of aluminum sulphate will produce the results desired.

If the lime turns out to be of no value, continue in the same manner with sodium carbonate, the various clays, and stone dust, and it is possible that any one of these inexpensive coagulant aids may turn out to be what is required.

If it happens that lime, clays, stone dust, etc. all turn out to be of no value in your raw water, then you still have all the proprietary coagulant aids and also activated silica to try.

In trying these various coagulant aids, including activated silica, remember to run many jar tests in which all of the beakers are dosed uniformly with the same dosage of aluminum sulphate (we have selected 2.0 grains purely as an example, and it could be any other figure), but in which the coagulant aids are added in a wide variety of dosage ranging as low as fractions of a part per million up to as possibly as high as 30 parts per million.

Particularly remember, when trying out coagulant aids, to run each jar test in duplicate -- in the first run apply the coagulant aid prior to the aluminum sulphate, and in the second run apply the coagulant aid after the addition of the aluminum sulphate. (There have been cases in which it was found desirable to add some of the coagulant aid prior to the aluminum sulphate, and the remainder after the aluminum sulphate).

Let us assume that of all the coagulant aids tried the activated silica is the best, and that with all the beakers dosed at 2.0 grains per Imperial Gallon of aluminum sulphate, and varying amounts of activated silica in the various beakers, the one containing 10 ppm of activated silica and 2.0 grains of aluminum sulphate is the very best, then we can say that you have established a suitable activated silica-aluminum sulphate ratio for your raw water.

Then you should run another jar test in which the dosage for the six samples in the beakers appears as follows:

Beaker No.	1	2	3	4	5	6
Activated Silica as parts per million	2.5	5	7.5	10	12.5	15
Aluminum Sulphate Grains per Imp. Gal.	0.5	1.0	1.5	2.0	2.5	3.0

--- in this manner you can raise or lower the aluminum sulphate dosage to meet high or low turbidities, but always keeping the ratio of activated silica to aluminum sulphate (as found in jar tests) constant. Your speaker does not

agree with those persons who assume that one fixed dosage of activated silica is all that is required in any water any more than a fixed dosage of aluminum sulphate is likely to be always suitable. When using activated silica, raise or lower it in ratio with the aluminum sulphate dosage as required.

THE PROOF OF THE PUDDING IS IN THE MAKING

The above is about all one can tell a group of students about jar tests, to do more would be like trying to describe a sunset to a blind man -- you must take this equipment in hand and run quite a number of jar tests on your own water; you will find it a most interested and rewarding enterprise.

As you become more experienced you may find it desirable to run the turbidity, colour, pH and alkalinity on the settled water in the beakers, but we will not go into detail on this aspect here, since the primary purpose of this lecture is to merely familiarize you with the basic jar test.

After running some of these jar tests, if you run into some confusing findings, you can obtain free help and advice from the technical personnel of the people who sell activated silica, the proprietary coagulant aids and the primary coagulants. Do not forget that a request to your District Engineer can provide you with technical personnel for a day or two from the OWRC to assist you in this work.

SOME PRACTICAL CONSIDERATIONS

Stirring Time

The procedure specifies exactly 15 minutes stirring at 30 RPM. This arbitrary figure is only of value when two water plants at some distance from each other wish to compare research on coagulants and coagulant aids. The 15-minute stirring will be too short in a modern plant where the flocculation period is 40 to 60 minutes and too long in an old plant where only 5 minutes flocculation is provided. Increase or decrease this stirring time to match your plant conditions including the shortened or lengthened stirring times brought about by high or low pumpage, or one flocculation basin being out of service for cleaning or repair.

Variation in Settling Period

A 5-minute settling period can be useful in supporting a decision made just at the end of the stirring period. For example, the sample may show promise during stirring, but after the stirring is stopped the floc may not settle in 10 to 20 minutes. On the other hand, a sample with a larger dosage, and having exactly the same appearance during stirring, will start to settle quite well in less than 5 minutes and will quickly show a "skin" of approximately a half inch of clear water at the top of the beaker -- the larger dosage would generally be preferable under such circumstances.

Check of Plant Efficiency

What happens if you produce perfect results in the jar tests and yet results in actual plant operation are anything but perfect? Obviously, your plant has some physical shortcomings. Using jar tests to help uncover deficiencies in plant design, a first series of experiments is run to determine not only the correct coagulant, but also the optimum dosage of that coagulant.

A second series is then run in which the optimum plant coagulant dosage is added to all six beakers and single beakers are taken out of the stirring machine at 5 or 10 minute intervals, over a 45-minute span (longer, if necessary) to find the optimum stirring period.

Using the optimum stirring period found in the second series, a third series involving varied coagulant dosages is run to see if a more economical coagulant dose can be used with the new stirring time. In such work, it is often wise to plot the original raw water turbidity plus the settled turbidity found in the various beakers for various stirring times on graph paper, the reason being is that a jar test is a thing of the moment, and it is something that is thrown down the sink within hours, whereas graphs can be exhibited to your consulting engineers to assist them in improving your plant.

Such experiments can reveal shortcomings in the flash mixing (if your plant is lucky enough to have it), coagulation, flocculation, and settling processes of your plant, and the need for introducing such improvements as more accurate feeders, better mechanical mixing and flocculating equipment, and longer times for mixing, flocculating and settling.

It is worth noting that long settling basins are desirable, but they are worthless if the original acts of flash mixing and flocculation are inadequate. Any conclusions drawn from a series of jar tests should only be used in conjunction with the particular raw water, and the particular water treatment plant, since generalizations about all water plants based on the findings at one plant can do more harm than good.

GENERAL OBSERVATIONS

When the coagulant and coagulant aid is added to the stream of water, it should not be permitted to stream into the water in a quiescent part of the operation, but should be added to a point where great turbulence and, therefore, instantaneous mixing is assured. Thus, it is a very safe generalization to say that in modern plant design a power driven flash mixer is very much worthwhile.

Another safe generalization borne out by many observations, based on new plants and old plants that have been renovated, is that after the flash mixing there should be a period of slow mechanical stirring by means of power driven mechanical flocculators for a period of not less than 30 minutes up to as long as 60 minutes. Non-power driven mechanical flocculation of the type provided by short 5-minute contact over and under baffles are not worth the sticks of dynamite required to blow them up. Your speaker saw a set of mechanical flocculators pay for themselves in aluminum sulphate savings in an 18-month period.

Another generalization that is fairly safe to make is that the longer the settling time after good flocculation, then generally the better the plant results. With the new multi-media filter beds coming into wider use, there has been some discussion relative to lesser settling times, thus permitting these new filter beds "to take it on the chin" in removing fairly high turbidities. It is all right to let these new filter beds take it if conditions dictate, but remember they are only filter beds and, therefore, the lower the influent turbidity, due to good flocculation and adequate settling, the longer these filters will run prior to requiring backwash.

The new types of filters are designed to give a clearer water at a much higher rate per unit area and to absorb rough spots in plant operation, but you should not continually abuse them and, therefore, the best water plant practice is to make sure that all facets of your operation are beyond criticism. Thus your speaker calls for the very best in flash mixing, flocculation and settling to be followed by the very best in filtration methods.

RADIOACTIVITY AND WATER TREATMENT

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INTRODUCTION

The lecture will deal with radiation hazard as related to nuclear attack and peacetime radiological water pollution. In the section on nuclear attack consideration is given to operator protection, contamination of supplies, and the decontamination of water.

There is much written about nuclear weapon effects. Water supply facilities or operators near a nuclear detonation will be so seriously affected by blast and shock, thermal radiation, and initial nuclear radiation that operation will be impossible. Also, a fourth factor, residual radiation, can seriously affect distant areas. However, with proper planning for operator protection and standby power, the facilities not immediately damaged by a nuclear detonation may continue to function effectively.

In addition to operator protection from radiation, the ability of the water utility to function in the face of a power failure, caused by the destruction of transmission and generating facilities, must be considered. Without electric power many systems could only supply water for a short time from elevated storage. Assuming that standby generating equipment is available, the reliability of fuel supply in the face of residual radiation and driver protection should be investigated.

NUCLEAR ATTACK

In the event of a nuclear attack, the waterworks operators beyond the zone of total destruction will be faced with the protection of himself against external gamma radiation and the protection of the public due to contamination of the water supplies by fallout. The gamma radiation to the operator will be the major hazard during the early post attack period; however, with time the external hazard diminishes but the internal hazard may become of greater concern due to ingestion

of radioactive materials in the contaminated water. Because of this, two basic standards are necessary for the waterworks operator, one for personnel and the other for radioactive contamination of the water supplies.

Operator Protection

Allowable Exposure

The external hazard due to gamma radiation after a nuclear explosion is very significant since the human body is susceptible to damage from these deeply penetrating rays. If we are exposed to large amounts of radiation in short periods of time, sickness and death may result. Few of us may become seriously sick when exposed to 100 roentgens, whereas majority of us receiving a short-term dose of 600 roentgens would die. Exposure to 300 roentgens would probably cause radiation sickness in some but death in only a fraction of us.

Control of Exposure

During the immediate post attack period when the external radiation field is high, waterworks personnel may be called upon to perform duties that will involve deliberate radiation exposure. The radiation dose associated with these duties can be controlled if the radiation field is measured and the length of exposure adjusted accordingly. Record of the doses received by the personnel should be maintained to keep the total exposure within the personnel standards.

Standard radiation survey meters can be used to determine the external hazard associated with these duties. Film badges, dosimeters and pocket chambers are also very useful in measuring the radiation dose received by the individual. However, the film badge would not be very useful in an emergency since the film requires processing before results can be obtained. The dosimeters and pocket chambers would be suitable since they can be read directly.

By using the above instruments and keeping records of the individual radiation doses, the waterworks operator can protect himself from the external gamma radiation by keeping his accumulated radiation dose within the personnel standards.

Shelters

When not working the operator should be sheltered from radiation. A layer of concrete eight inches thick will provide an attenuation factor of ten; doubling the thickness will increase the factor to 100. Thus each extra eight inches of concrete will increase the protection factor tenfold. It should be remembered that radiation will come from any direction where radioactive particles have settled; platforms, rooftops, ground, etc.

In most instances the safest place for an operator and his family during and after the fallout period will be the water treatment plant. With a minimum of development most plants can provide an attenuation factor of up to 100 whereas the average home may provide a factor to 2 - 10.

Radioactive Contamination of Water Supplies

A simple and rapid method is required for the measurement of radioactive contamination of water supplies under emergency conditions since laboratory facilities and elaborate counting equipment may not be available. One such method has been developed by the Atomic Energy of Canada at Chalk River. In this method standard radiation survey meters are used to measure the gamma radiation associated with a suitable volume of water. The magnitude of the gamma field has been correlated with readings obtained from water which contained the permissible concentrations of fallout products of known age. Based on this work, guidelines were established for the maximum radioactivity in the contaminated waters. These guides along with the points of measurement for the typical water supplies are listed in Table I.

TABLE I

GAMMA FIELD MEASUREMENTS FOR MAXIMUM PERMISSIBLE
WATER CONTAMINATION

This table applies to any fresh fallout and is for ten-day consumption. Values should be halved for 30-day consumption.

<u>Water Body</u>	<u>Time Since Bomb Exploded</u>			
	<u>12 hrs.</u>	<u>1 day</u>	<u>2 days</u>	<u>10 days</u>
Reservoir or lake, measured far from shore	100 mr/hr	50 mr/hr	25 mr/hr	12 mr/hr
Reservoir, pond, etc., measured at arms length from shore, close to sur- face and over water at least 2 ft. deep	50	25	12	6
Water tank, 150 to 1000 gallons, measured in contact with centre of one surface	50	25	12	6
Water can, 2 to 4 gallons, measured in contact with centre of one face	25	12	6	3

mr/hr - milliroentgens/hour

Table I can be used for both treated and untreated water although in the case of fallout older than two days and water that has been undergone unusual chemical treatment, a day should be allowed to lapse between treatment and final measurement of the radioactivity. It should not be used with water which has been contaminated with old fission products in which case recourse should be taken to a full radiochemical analysis.

The above method may not give results which are extremely accurate but it is reliable enough to point out dangerous levels of radiation under emergency conditions. Wherever possible services of other agencies such as the Radiation Protection Branch Laboratory of the Department of Health, laboratory facilities at Universities and research institutes which have more sensitive instruments should be solicited for more accurate analyses.

Decontamination of Radioactive Water

The radioactivity in open water due to nuclear fallout would be made up of many radionuclides, some insoluble, attached to the fallout dust and some soluble. Most of the insoluble radioactivity can be removed by conventional treatment such as settling, coagulation and filtering. The soluble radioactivity, amounting to five to ten percent of the total is more difficult to remove. Distillation and ion exchange are two possible methods of removing the soluble radionuclides.

In most cases well water will be the first choice in a nuclear emergency. When well water is not available, the next choice will be water receiving the best treatment for the removal of particulate matter. Here again the degree of contamination and the depth of water affording dilution are to be considered. In any case water is an essential part of the human diet and any water is better than dying of thirst. Also, as water is required for many other purposes beside drinking, the supply can not be completely shut down because of contamination.

PEACETIME RADIOLOGICAL WATER POLLUTION

As discussed earlier the major hazard associated with a nuclear explosion is the external radiation field while internal exposure is of minor importance. This is because the radioactive elements produced by a nuclear explosion are relatively short lived and occur in large

quantities thereby producing a short lived radiation field of high intensity. In this case, the alpha activity is not taken into consideration since its external hazard is negligible. On the other hand, when dealing with peacetime radiological water pollution, we are concerned with internal ionizing radiation due to much smaller quantities of long-lived radionuclides. Some of these are naturally occurring elements such as Carbon 14, radon, uranium and thorium and all of their daughter products. Some are man-made, such as Strontium, Cesium, Iodine, barium, lanthanum, etc., which have resulted from peacetime nuclear weapons tests, nuclear reactors and other uses of atomic energy. Even the alpha emitters must be considered carefully since the alpha particle is capable of producing internal damage.

Many of the elements tend to accumulate in specific organs within the human body and thus provide localized radiation to that organ. For example, Radium 226 concentrates in the bone while Iodine 131 accumulates in the thyroid. If concentrations of such elements are allowed to become excessive biological damage may be caused to the organ.

Several national and international agencies composed of authorities in the radiation protection field have set up guide levels for the maximum concentrations of the various radionuclides in drinking water. A list of names of the major agencies is appended for your information and copies of their guidelines are readily available. It should be remembered that the limits quoted in these guides are based on a lifetime exposure and exceeding these levels for relatively short periods of time does not necessarily produce a great hazard.

At the present time, we do not have any serious radiological water pollution problems in Ontario with the exception of wastes from the uranium mining industry. However, with the increasing use of atomic energy, we may be faced with this problem in future in addition to the other existing types of water pollution.

APPENDIX

References for Agencies Which Have Set Up Guidelines for Protection Against Radiological Pollution

1. International Commission on Radiological Protection, Report No. 6 Pergamon Press, New York, N.Y. - 1962
2. U.S. Public Health Service. Drinking Water Standards, PHS Publication No. 956, Revised 1962. Superintendent of Documents, U.S. Government Printing Office, Washington D.C. 20402 - March 1963
3. World Health Organization - International Standards for Drinking Water, Second Edition, Geneva (1963)
4. National Committee on Radiation Protection. Maximum permissible body burdens and maximum permissible concentrations of radionuclides in air and water for occupational exposure, NBS Handbook 69, U.S. Department of Commerce.
5. Federal Radiation Council. Background material for the development of radiation protection standards, Report No. 2. Superintendent of Documents, U.S. Government Printing Office, Washington D.C. 20402 (September 1961)

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Civil Defense Report of Waterworks Operators
FG- F 3.6 June 1966
2. Tait G.W.C. and Merrit W. F.
Emergency Radiation Monitoring of Drinking Water
CRNP - 733 AECL No. 505
Atomic Energy of Canada Limited, Chalk River Project,
Research and Development - October 1957.

- E -

MATHEMATICS REVIEW

by

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ONTARIO WATER RESOURCES COMMISSION

MATHEMATICS REVIEW

1. Add:

(1) $\frac{1}{8}$, $\frac{3}{16}$ and $\frac{1}{4}$

(2) 5.68, 0.233 and 75.2

(3) $2\frac{1}{4}$, $3\frac{1}{2}$ and $7\frac{3}{8}$

2. Multiply:

(1) $3\frac{3}{8}$ by $\frac{3}{5}$

(2) 68.3 by 6.23

(3) 0.023 by 38.7

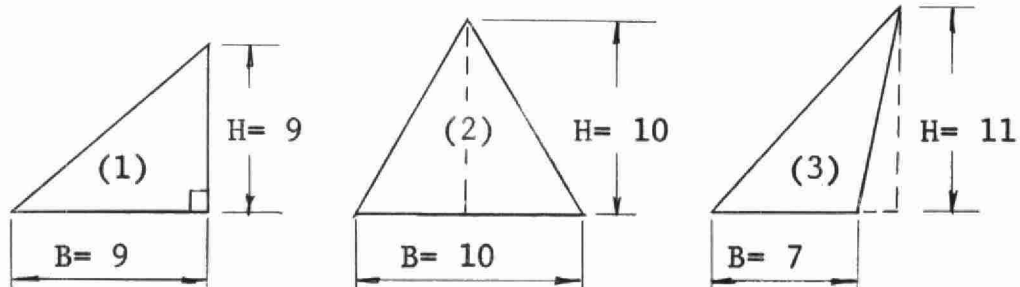
3. Divide:

(1) $\frac{5}{6}$ by $\frac{2}{3}$

(2) 425.509 by 68.3

(3) 5481 by 63

4. Find the area of the following:



Note: In triangle (2) the two angles at the base and the two sides are equal. All measurements are in feet.

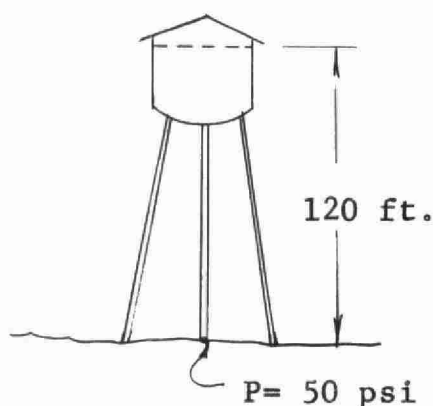
5. What is the area of a rectangle whose dimensions are 24 inches by 12 inches

- (a) in square inches?
- (b) in square feet?

6. What is the volume of a tank whose dimensions are 30 feet by 15 feet by 10 feet deep

- (a) in cubic feet?
- (b) in gallons?

7. What is the area, in square feet, of a circle which has a diameter of 5 feet?
8. Find the volume of a cylinder which has a diameter of 6 feet and a height of 11 feet
 - (a) in cubic feet
 - (b) in gallons.
9. A tank which has a capacity of 60 gallons contains 25 gallons of water, 15 gallons of liquid alum and 20 gallons of lime slurry. What per cent of the total does each of the three substances in the tank represent?
10. The water pressure at the bottom of an elevated storage tank is 50 pounds per square inch (psi). If the tank has a height of 120 feet is there sufficient pressure to fill it?



11. How many pounds of gas chlorine are required to disinfect 1,500 feet of 8-inch-diameter watermain if the required dosage is 50 parts per million?
12. If the chlorine compound used in question 11 has only 70 per cent available chlorine how many pounds are required?
13. A rectangular settling tank has a volume of 6,000 cubic feet. How long would it take to fill the tank with a pump rated at 750 gallons per minute?

14. The velocity of water in an 8-inch-diameter pipe is 1.5 feet per second. What is the flow
- (a) in cubic feet per second?
 - (b) in gallons per minute?
15. A pump has a capacity of 500 gallons per minute. If the discharge is to receive a chlorine dosage of 0.5 parts per million what must the capacity of the chlorinator be in pounds per day?

CROSS-CONNECTIONS AND BACK SYPHONAGE

P. G. Spenst, P. Eng.,

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Division of Sanitary Engineering

- OUTLINE - (1) Definition of terms
(2) Health considerations
(3) Technical considerations
(4) Regulations past and current
(5) Methods of preventing back flow
(6) Common cross-connections
(7) Recent experience with cross-connections
(8) Some case histories
(9) Conclusions

DEFINITION OF TERMS

"Cross-Connection"

A "cross-connection" shall mean any actual or potential connection or structural arrangement between a public or consumer's potable water system and any other source or system through which it is possible to introduce into any part of the potable system any used water, industrial fluid, gas or substance other than the intended potable water with which the system is supplied. By-pass arrangements, jumper connections, removable sections, swivel or change-over devices and other temporary or permanent devices through which or because of which "back flow" can or may occur, are considered to be cross-connections.

"Back Syphonage"

"Back Syphonage" shall mean a form of back flow due to a negative or sub-atmospheric pressure within a water system.

Note:- Above definitions from "Manual of Cross-Connection Control Recommended Practice" - (University of Southern California).

HEALTH CONSIDERATIONS

Man in his normal environment is vulnerable to insidious attack by many elements which may unknowingly be contained in the water used to supply his personal needs.

It is not the intention here to detail or even name all the ailments that may arise from a supply of impure water but merely to point out that the hazard does exist. If details or proof are required, these are available from medical records.

The fact that these undesirable agents may be invisible, tasteless and odourless makes them difficult to detect and it becomes increasingly important that once a supply of water has been rendered potable that every effort be made to safeguard it and keep it potable.

The purpose of this lecture is to consider the ways in which a potable supply of water can become contaminated and the methods available to guard against such contamination.

TECHNICAL CONSIDERATIONS

A cross-connection, as we have defined it, is not in itself injurious to health. It is, however, a weak link in the armour of a potable water system. A place where the undesirable elements mentioned above can gain easy access to the supply of potable water.

It is somewhat analogous to a case of dynamite stored in the basement of your home. It may be there for years and do absolutely no harm. It is, however, a constant threat which, under the right circumstances, can become lethal and destructive without notice or warning.

In the case of the cross-connection, the only additional requirements are a reversal of pressure causing a foreign substance to enter the potable water system. If the foreign substance is something injurious to health, then illness and loss of life are apt to follow.

REGULATIONS - PAST AND CURRENT

In view of the dangers already outlined in association with cross-connections, I believe it may be reasonable to assume that the fight against cross-connections is probably as old as public or communal water systems.

I have no real ancient data on cross-connections but I have some Province of Ontario documents dated 1921 and they seem to indicate that cross-connections posed the same problems in those days as they do to-day. The problem is not new, but it is more important than ever before because every day more and more people depend on a public system for their supply of potable water.

People have at various times enacted legislation outlawing cross-connections, but this has not proved to be a solution to the problem. Things for which there is a demand cannot be legislated out of existence. In this respect cross-connections are like liquor, they refuse to disappear just because they are illegal. Let us not fool ourselves that we are going to stamp out cross-connections over night for it is not likely to happen in this generation or in the next.

Knowing that we do have cross-connections and will, in all likelihood continue to have them, we are making an attempt to regulate and control them and make them as safe as possible. With this in mind, I refer you to the following sections of Regulation 471:

Sections 2; 3; 24; 27; 28; 41; 43; 44 and 153.

METHODS OF PREVENTING BACK FLOW

Air Gap

Air gaps in water distribution piping are considered by some people to be the only positive protection against back flow. That a proper air gap will protect a water system is true enough. It loses its reliability, however, because it is so easily bypassed and once this is done, it usually affords no protection at all.

It has another disadvantage in that it usually completely depressurizes the water often necessitating costly re-pumping.

Barometric Loop

A barometric loop is simply a vertical loop of pipe so high above the maximum water level on the downstream side that water cannot be forced back over the loop by atmospheric pressure.

Such an arrangement has the advantage that water need not be depressured and yet will not reverse by syphonic action.

It loses all protective value, however, if the downstream side is subject to pressures other than atmospheric.

Vacuum Breaker

A vacuum breaker offers about the same degree of protection as a barometric loop. Its advantage is less room required and usually a lower cost price. It is, however, subject to mechanical failure in addition to its inherent limitations.

Check Valves

Where the discharge openings of a water distribution system are submerged and the back pressures are apt to be above atmospheric, back flow cannot be prevented by barometric loops or vacuum breakers and a check valve arrangement of some kind is usually resorted to.

A single check valve when functioning as intended, will prevent back flow. Check valves are, however, subject to mechanical failure and for that reason a single check valve in a supply pipe is not considered adequate to reduce the hazard to an acceptable level.

If two check valves are installed in series there is still a chance that both may fail but the mathematical probability of both valves failing is less than the probability of one valve failing.

If we further add two gate valves, one before and one after the check valves so the gate valves may be used to isolate the check valves for service and maintenance, the degree of protection is further improved.

The next step is to install test cocks between valves so they may be tested to know when repairs or maintenance are required.

Reduced Pressure Zone Back Flow Preventor

The reduced pressure zone back flow preventor consists of the double check and double gate valve arrangement described above with further refinements incorporated. The reduced pressure zone is the volume enclosed between the two check valves. The reduced pressure is obtained by spring loading the first check valve (in direction of normal flow). In normal operation the water pressure in the reduced zone would always be less than the supply pressure by the amount of the spring loading. Should there be a pressure drop in the supply or a pressure increase on the delivery side with accompanying leakage through the downstream check valve, the reduced pressure zone is kept below the supply pressure by bleeding to atmosphere.

The drain valve from the reduced pressure zone to atmosphere is controlled by a spring-loaded diaphragm. One side of the control diaphragm is subjected to supply pressure and the other to pressure in the reduced zone. The supply pressure tends to close the drain valve. The pressure in the reduced zone plus the spring pressure tend to open the drain valve.

In this way, assuming the worst possible situation, namely, all valves leaking, a back-up of water through the downstream check valve would be drained to atmosphere before the pressure in the reduced zone would build up high enough to force water back through the upstream check valve.

COMMON CROSS-CONNECTIONS

- (a) Any water faucet with a hose connected which can reach a contaminating fluid.
- (b) A submerged discharge such as a flushometer water closet.
- (c) Direct connected trap seal primers
- (d) Fire protection systems cross-connected with potable water systems (sprinkler system).
- (e) Fire hydrants cross-connected with polluted ground water.
- (f) Water operated devices such as ejectors, aspirators, sprayers, aerators.

SOME RECENT EXPERIENCE WITH CROSS-CONNECTIONS

- | | | |
|--|---|------------|
| (a) Ford & Chrysler Cupola |) | |
| (b) Lake Superior pulp mill |) | Discuss as |
| (c) North York Apt. Building and other heating plant problems. |) | time |
| (d) East York spot check |) | permits |
| (e) Watts Regulator No. 8 & C-9 |) | |

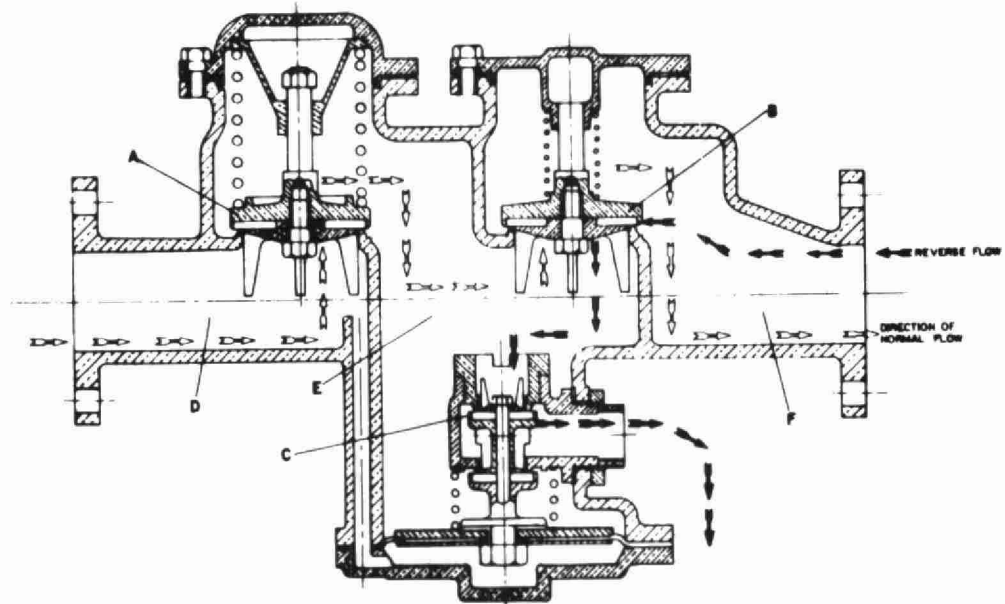
SOME CASE HISTORIES

Discuss as time permits some of the cases documented in U.S.A. data.

CONCLUSIONS

In conclusion, no panacea for cross-connections can be offered. It is hoped all water works operators will realize that cross-connections exist all around us and that they are probably here to stay. They are a continual threat to life and health and should be regarded as such. They should be reduced in number to an absolute minimum by eliminating all unnecessary and unprotected cross-connections. Cross-connections which cannot be eliminated should be kept in plain sight rather than hidden away and they should be protected against back flow by the best possible method for the conditions prevailing.

If the probability of contaminating a potable supply through a cross-connection cannot be reduced virtually to zero, then the cross-connection should be eliminated regardless of financial expenditure required.

REDUCED PRESSURE BACKFLOW PREVENTER

The Beeco Model 6-C Reduced Pressure Backflow Preventer contains the following operating parts:

- 2 Spring-Loaded Vertical Check Valves (A and B)
- 1 Spring-Loaded, diaphragm actuated, Differential Pressure Relief Valve (C)

Check Valve A is spring-loaded to a closed position, and causes all water passing through it to be automatically reduced in pressure by approximately 8 pounds per square inch.

Check Valve B, which forms the "double check" feature of the device, also acts to prevent unnecessary drainage of the domestic system in case a backflow condition occurs. This valve is lightly spring-loaded and, therefore, very little pressure reduction is made in passageway F and in the pipe lines beyond.

Relief Valve C is spring-loaded to remain open, and diaphragm actuated to close--by means of differential pressure.

To illustrate the operation, we will assume water, having a supply pressure of 60 psi, is flowing in a normal direction through the device (as shown by the white arrows). If we close all valves beyond Area F, creating a static condition, the water pressure in Area D will be 60 psi and the water pressure in Zone E will be 52 psi.

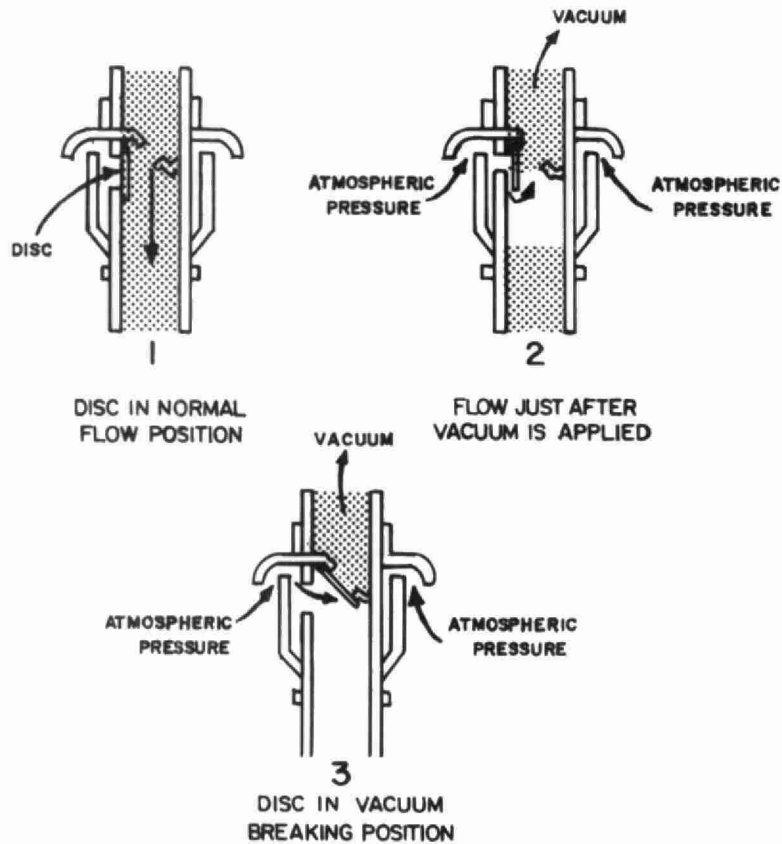
The inlet pressure of 60 psi is transmitted through a cored passageway to the underside of the diaphragm of Relief Valve C. This valve is spring-loaded to remain in an open position until the differential pressure amounts to 4 psi or more.

Therefore, during normal operation, the 8 psi differential pressure produced by Check Valve A exceeds the spring-loading of Relief Valve C and causes Valve C to remain closed.

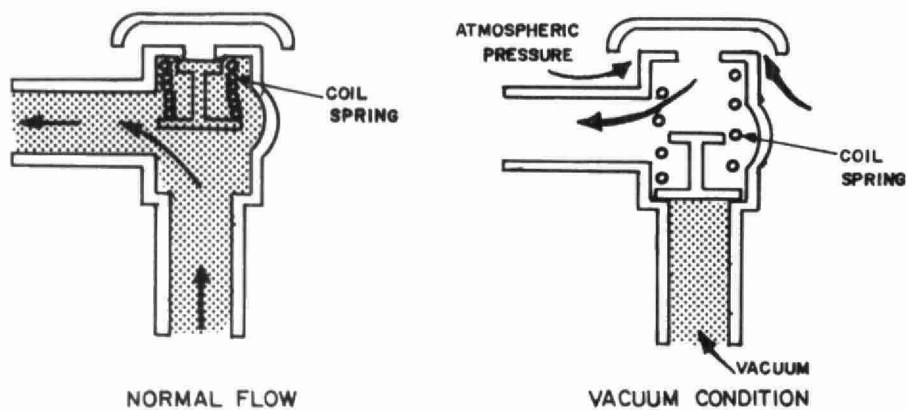
There are two conditions that tend to produce backflow:
(1) decrease in pressure in the supply line, and (2) increase in pressure in the discharge line, or domestic piping system.

OPERATION OF A VACUUM BREAKER

(1) SWINGING DISC TYPE



(2) SLIDING SPOOL TYPE



NOTE: This shows the principle of operation. Other types have a sliding spool which closes the air inlet when there is forward flow.

HYDRAULICS OF WATER FILTRATION

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INTRODUCTION

The flow of water through a graded bed of sand and gravel is a very complicated mechanism which continues to defy analysis because of the many factors involved.

The formulae and methods proposed to date have been based on physical filtration theories which deal with media size, filtration rate, and water temperature. Even the most modern and best of these contain a number of parameters which must be determined on a full-scale or model filter by observation to obtain their solution. It is, therefore, not possible to predict results in advance without first conducting field or laboratory experiments.

More recently, theories which consider the chemical characteristics of the aqueous phase and the surface characteristics of the suspended particles and filter media have been given more attention. However, to date, they have not been expressed in mathematical terms permitting prediction of filter performance.

A detailed discussion of these theories of filtration and the factors affecting filtration rates is beyond the scope of this lecture.

The hydraulic features of the conventional rapid sand filter are discussed in sufficient depth that one should be able to understand what is happening in the filter while filtering. These are rate of flow controllers; flow through clean sand and gravel; head loss during filtration; and filter bed hydraulic profiles. Examples of typical hydraulic calculations are not included, however, because of their length and complexity.

FILTRATION

A filter should be operated at a rate which results in maximum production of quality water per filter run and, therefore, in minimum backwash percentage. It can be identified as the lowest rate at which the head loss curve becomes most nearly linear. This is possible when filtering suspensions have a "strong" tendency to be removed at the surface.

The particles have adequate strength to resist the hydraulic shear forces tending to wash them down into the filter. The significance of the rate of head loss will be explained in more detail later under "Head Loss During Filtration".

Until recently, the maximum filter rate for any rapid sand filter was considered to be 2 USGPM/Ft.². In the last few years with better pre-treatment methods and new filter media, higher filter rates, as high as 4 to 5 USGPM/Ft.² have been used successfully. The matter of utilizing "higher" rates at existing 2 USGPM/Ft.² plants requires careful consideration because of the many factors involved. The hydraulic factors include: (1) available loss of head at any critical point; (2) capacity of rate controllers; (3) capacity of underdrain systems and effluent piping and; (4) washwater trough overflow rates. All factors should be evaluated by experimental testing before re-design for higher rates is undertaken.

RATE OF FLOW CONTROLLERS

As the time of operation of a rapid sand filter increases, the loss of head increases through the filtering media. If a uniform rate of flow through the filter is to be maintained, the size of opening in the effluent pipe must be controlled to offset this increasing loss of head. This is done by installing a rate of flow controller in the filter effluent pipe to impose a head on the system when the media is clean and to remove this head as the filtered material accumulates in the filter bed.

Rate of flow controllers all operate on the same basic principle of using a differential producer to operate a throttling mechanism. The differential producer is usually a venturi tube section. The throttling devices employed are various such as butterfly valves, balanced valves, and specially designed units. The power to manipulate the valves may be hydraulic, pneumatic, or electric.

Venturi Tube (Figure 1)

A typical venturi tube is shown in Figure 1. An explanation of how it works together with the derivation of its discharge formula is given in the appendix.

Counterweighted Lever Arm Rate Controller (Figure 2)

This is a direct acting controller where the different pressures at the throat and outlet of the venturi section are transmitted to the two sides of a diaphragm pot or piston chamber. These differential pressures are reflected directly on the diaphragm or piston moving a distance dependent upon the difference in pressure exerted. This movement is transmitted mechanically to a counterweighted scale beam.

The controller works on the theory involving the balance of moments as created on the controller beam. The balance of forces is created by setting the counterweights at the pre-determined distance from the fulcrum thus balancing the venturi differential applied to the diaphragm pot. This position of balance is a function of the valve opening. The control valve uses statically and hydraulically balanced blades, mechanically linked with the diaphragm pot.

Power Operated Rate Controller (Figure 3)

Indirect acting rate controllers operate through a pilot valve connected to the venturi tube and also to an outside source of power. In these, the pressure differentials of the venturi tube actuate the pilot valve which directs the application of the separate power source to the throttling device. These controllers function as amplifiers to convert the relatively small energy from the venturi differential into a powerful operating force.

Air operated rate controllers have these three basic elements: (1) a regulated air supply at a specific pressure; (2) one or more converters which convert differentials to a linear controlled air pressure and; (3) a mechanism operated by the controlled air pressure for actuation of the air operated valve of the control device.

FLOW THROUGH CLEAN SAND AND GRAVEL BEDS

The filter media offers tremendous resistance to the flow of water through it. A bed of clean sand and gravel, therefore, has an initial head loss which is the head required to produce a given flow through it.

Theory

In previous lectures we have indicated to you how the Law of Continuity and the Law of Conservation of Energy have been used to derive mathematical formulae to describe the flow of water through pipes and conduits. An analysis of the problem compared to flow in pipes considering only the physical factors yields an expression which indicates that initial head loss increases with an increase in rate of flow, and depth of filtering material; and with a decrease in grain size, porosity and water temperature. The expression $h_f = C \times \frac{VL}{D^2} \times \frac{u}{p}$ is explained in some detail in the appendix.

In comparing anthraflit with sand the former will give lower initial head loss than the latter when all other factors are constant. This is largely caused by the fact that anthraflit has porosity values 13 to 15 per cent greater than that of sand.

Determining Initial Head Loss in Practice

Because of the complicated interplay of variables involved head loss values are determined in practice directly by observation rather than by calculation.

The loss of head to any point in the filter box is determined from piezometer or manometer tubes connected through the filter box wall to the point in question and carried up through the filter gallery to the filter console above the water surface in the filter. The tubes may be polyethylene 1/4" to 3/8" diameter. The distance from the level of the water in the filter to the level of water in the tube is the head loss to the point of the connection. For the total filter head loss the piezometer tube should be connected to the centre line of the filter effluent pipe.

It has been found from studying many rapid gravity filter installations that the initial head loss through the sand is from 6 to 8 inches, through the anthraflit is 4 to 6 inches, through the gravel is about 2 inches, and for the total filter including underdrains the effluent trough or pipe is from 1 to 2 feet.

HEAD LOSS DURING FILTRATION

Once the filter is placed in operation the media becomes coated and the particulate matter removed from the water fills up the openings between the grains. With fine media for "strong" floc, the effective zone of filtration is mainly in the upper one or two inches of depth. For "weak" floc, the removal of the suspended matter takes place in successively lower zones of the bed. For the coarser media more commonly used to-day penetration is deeper regardless of the floc characteristics. As clogging continues the head loss increases until the filter must be shut down and washed. The total head loss, therefore, is the sum of the surface cake head loss and the head loss in the bed.

The large number of variables involved and the limited amount of knowledge concerning their effects, individually and collectively have precluded the development of any formula by which filter performance can be predicted accurately. An AWWA task group is presently working on this problem hoping to develop a "filterability index" with universal application.

Recent research in quest of the "filterability index" has, however, increased our understanding of the mechanism of filtering water through media beds. As a result, there has been a trend to coarser materials, to uniform media rather than graded media to increase porosity, to choice between round and angular grains, and to multi-media beds.

For mixed anthrafil and sand beds, the water first passes through a pre-determined depth of coarse, lighter anthrafil supported by a lesser depth of finer, heavier sand. The high porosity anthrafil permits the deep penetration necessary for low rate of head loss and the fine sand removes the finer suspended matter which might be passed by the anthrafil.

Rate of Head Loss

If two measurements of head loss in a filter are taken "t" minutes apart during filtration then the difference in these head losses divided by "t" represents the head loss increase per unit of time or "rate of head loss". The length of filter run is the criterion by which many operators judge filter performance. This variable in turn is a function of the average rate of head loss.

The head loss in the compressible surface cake increases exponentially as the filter run progresses. (Figure 4).

The head loss within the sand bed develops in a linear manner because the rigid matrix of the sand grains prevents compression of the deposited material.

Break Through

At the end of the filter run there can be a break through of suspended matter particularly if the floc is weak.

Break through may be anticipated by watching the relationship between the length of the filter run and the head loss; that is, the rate of head loss. If the head loss increases exponentially this is a good indication that no break through is occurring. (Figure 4). On the other hand, if the rate is constant throughout or is decreasing it is an indication of "weak" floc which may result in a break through if the run is not stopped soon enough. (Figure 4). A drop in head loss indicates that a serious break through has already occurred.

Maximum Permissible Head Loss

The maximum available loss of head through a given filter is determined by the vertical distance from the water level in the filter box to the water level in the clear well, assuming a submerged filter discharge.

The maximum desirable loss of head is determined by the designer in fixing the filter box depth and the operating freeboard of the filter. Most filter boxes are 10 feet deep. If the filter has a one foot freeboard and an 18-inch deep underdrain system then the maximum loss allowable is 7.5 feet based on no negative head in the sand bed.

The maximum permissible loss, therefore, should not create a negative head in the filter box as pressures below atmosphere permit the release of air with the problems of air binding.

FILTER BED HYDRAULIC PROFILES (Figure 6)

What we have said earlier on flow through clean beds and now on flow while filtering may be better understood by studying Figure 6 of an 11'-6" filter box with a series of piezometer tubes at various elevations.

At the start of a filter run with clean water the total head loss from the influent water level "A" to the bottom of the sand bed "E" is about 1.5 feet, and to the head of water in the effluent pipe "H" about 2 feet as shown by the solid line.

As the suspended matter is filtered out the friction in the bed is increased, to a larger degree in the top layers, but to some extent throughout the sand bed, until at the practical end of the filter run, the loss to a point just below the top of the sand bed "C" is about 4 feet, to the bottom of the sand bed "E" about 7 feet, and to the head in the effluent pipe "H" about 8 feet as shown by the line of short dashes.

If the filter run is extended, the head loss will continue to increase until the condition represented by the line of dots exists. While a small positive head exists at "G" an 8-inch negative head exists at "D" and a 2 foot negative head at "E".

Somewhat similar negative head conditions will occur if the influent water level "A, B" is substantially lowered, thus lowering all manometer readings the same amount. This also causes a negative head of about 8 inches at "E" as indicated by the line of short dashes.

HYDRAULICS OF WATER FILTRATION

APPENDIX

VENTURI TUBE

As shown in Figure 1, the venturi tube has a short, straight "throat" section preceded by a short, convergent inlet section followed by a longer, divergent outlet section. Transitions are by easy curves. The "throat" section forms a constriction in the pipe line.

With no flow, water in the piezometer pipes at inlet (1) throat (2), and outlet (3) will all stand at the free surface of tank (1).

When the valve is opened the difference in water level between the tanks will cause flow from tank (1) to tank (2). A part of the total head (H) will be converted to velocity head $\frac{V_1^2}{2g}$ causing velocity (V_1) at inlet section (1).

Since the same quantity must pass the inlet and throat at the same time ($Q_1 = Q_2$) and velocities vary inversely as the areas of the cross-section ($V_2 = \frac{V_1 A_1}{A_2}$), the contracted throat section will cause an increase in A_2 velocity head $\frac{V_2^2}{2g}$. Because of the gradual increase in cross-sectional

area between the throat (2) and the outlet (3) the velocity increase in the throat is given up with a resulting recovery of nearly all of that part of the pressure head (H_D) converted to velocity head. If the flow were frictionless all of this converted head would be recovered and pressure head hp_3 would equal hp_1 .

The discharge through the throat can be determined using Bernoulli's equation, measuring H_D , that is the difference between hp_1 and hp_2 , and knowing the venturi tube coefficient.

$$\frac{P_1}{W} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{W} + \frac{V_2^2}{2g} + Z_2 + F \quad (1)$$

where F is the friction loss from (1) to (2)

= b x H_D where b is a factor which depends on the smoothness and proportions of the venturi tube.

Further $V_1 = \frac{A_2 V_2}{A_1}$ or in terms of diameters

$$V_1^2 = \left(\frac{d}{D}\right)^4 V_2^2 \quad \text{where } d = \text{throat diameter} \\ D = \text{inlet diameter}$$

and equation (1) becomes

$$H_D = \frac{P_1}{W} - \frac{P_2}{W} = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} + b H_D$$

$$\text{since } Z_1 = Z_2$$

substituting further

$$H_D (1-b) = \frac{V_2^2}{2g} - \left(\frac{d}{D}\right)^4 \frac{V_2^2}{2g}$$

$$V_2^2 = \frac{H_D (1-b) 2g}{1 - \left(\frac{d}{D}\right)^4}$$

$$\text{or } V_2 = \sqrt{1-b} \sqrt{2g H_D} \bigg/ \sqrt{1 - \left(\frac{d}{D}\right)^4}$$

$$Q = V_2 A_2 = A_2 \times \sqrt{1-b} \times \frac{1}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{2g H_D} \\ = A_2 \times C \times \frac{1}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{2g H_D}$$

where $C = \text{discharge coefficient} \sqrt{1-b}$
 $= \frac{\text{actual quantity discharged}}{\text{calculated theoretical discharge}}$
 $= 0.96 \text{ to } 0.99 \text{ say } 0.98 \text{ average}$

FLOW THROUGH CLEAN SAND AND GRAVEL

An analysis of the problem compared to flow in pipes considering only the physical factors gives the expression

$$h_f = C \times \frac{VL}{D^2} \times \frac{u}{p}$$

where h_f = head loss

C = coefficient

V = average velocity of the water within the filter bed which in turn is inversely proportional to the porosity (P). The porosity can be approximated from testing representative media samples, but cannot be determined precisely because of the amount of packing which takes place in the full-scale filter. The amount of packing in turn varies with grain shape, rate of flow and the time taken to reach the required rate of flow.

L = pipe length or depth of the bed as representing the length of a string of connected pores

u = viscosity which decreases with rising temperature

p = density which has a negligible decrease with rising temperature

D = pipe diameter which can be replaced by grain diameter. The grain diameter can be used only if its value remains relatively constant which is usually not the case. However, since the bed undergoes some degree of hydraulic grading, the formula can be applied to each layer in which size variation is not significant.

HYDRAULICS OF WATER FILTRATION

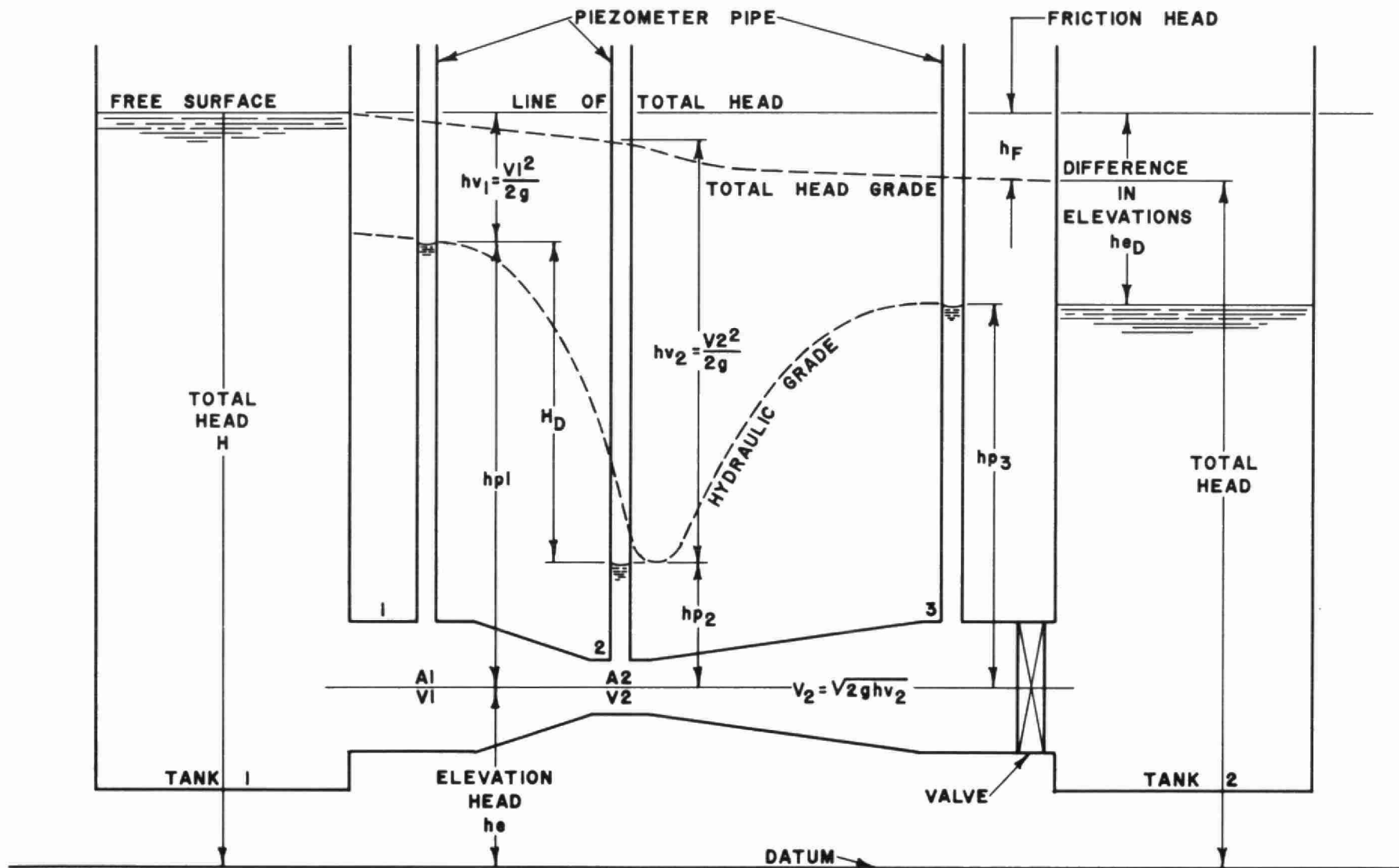
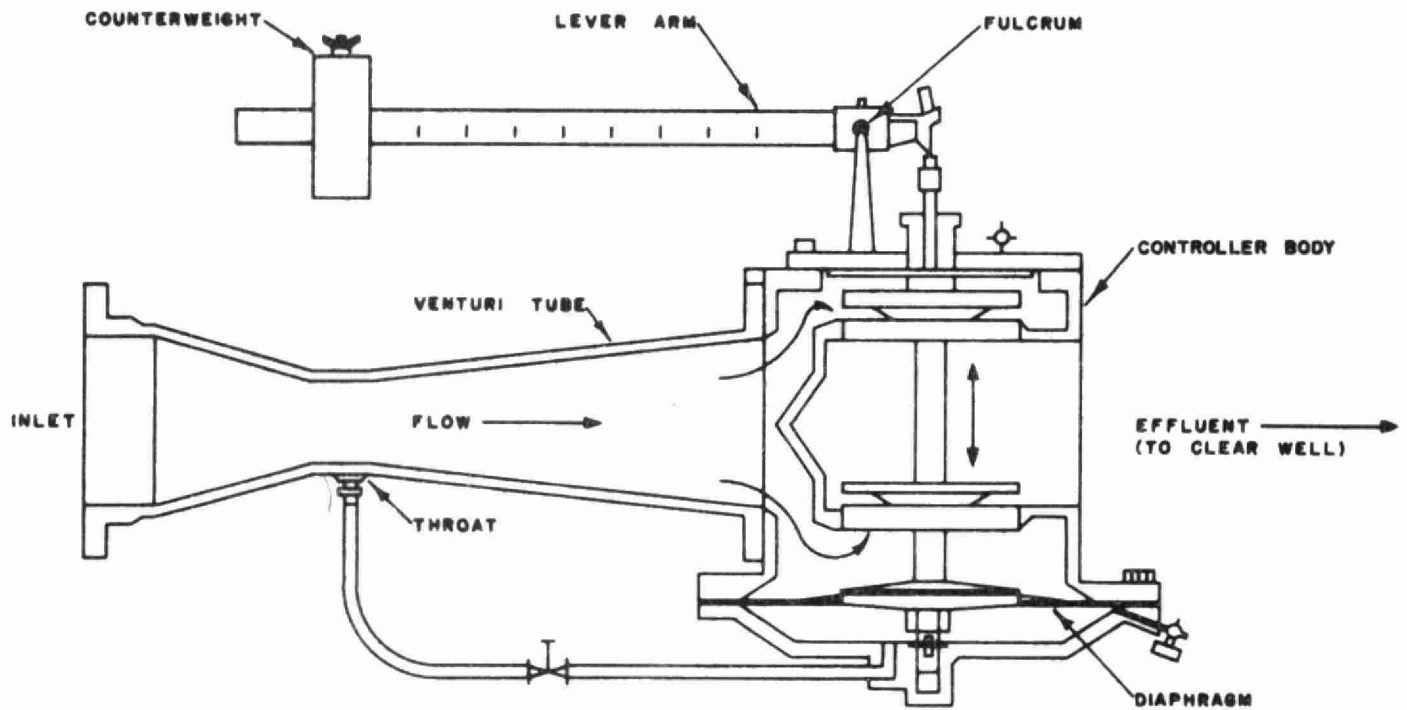


FIG. 1 - WATER UNDER PRESSURE FLOWING THROUGH VENTURI METER

HYDRAULICS OF WATER FILTRATION

**COUNTERWEIGHTED LEVER ARM RATE CONTROLLER.**

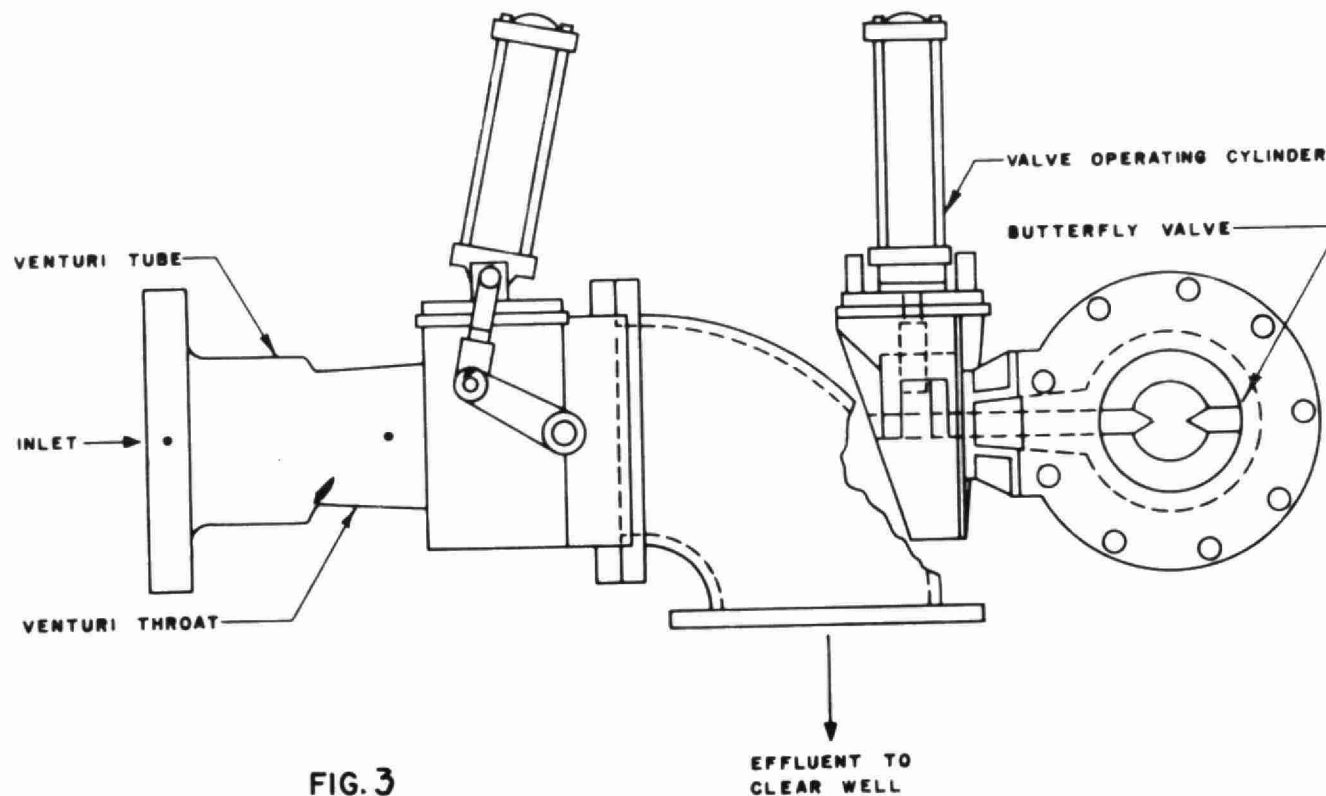
Position of counterweight on lever arm determines rate at which flow through filter is maintained. Differential produced across venturi tube is applied to diaphragm, low pressure (from venturi throat) to underside and high pressure (from venturi inlet) to top of diaphragm.

FIG. 2

HYDRAULICS OF WATER FILTRATION

SIMPLIFIED SKETCH OF POWER OPERATED RATE CONTROLLER

VALVE OPERATING CYLINDER IS CONTROLLED THROUGH PILOT VALVE OF
PNEUMATIC FILTER CONTROLS (NOT SHOWN)



HYDRAULICS OF WATER FILTRATION

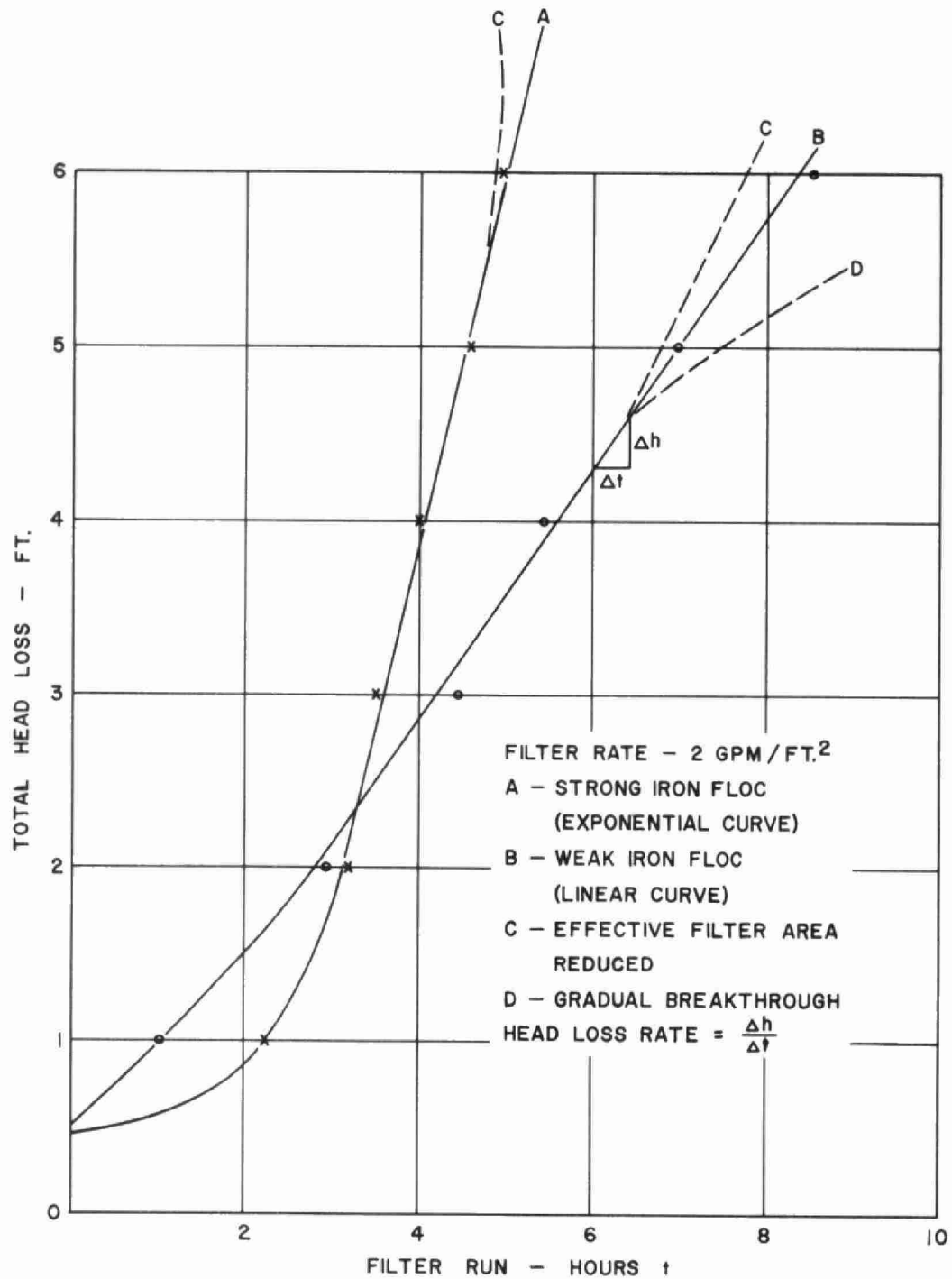


FIG. 4 - TYPICAL HEAD LOSS CURVES WHILE FILTERING DIRTY WATER (LABORATORY FILTERS)

HYDRAULICS OF WATER FILTRATION

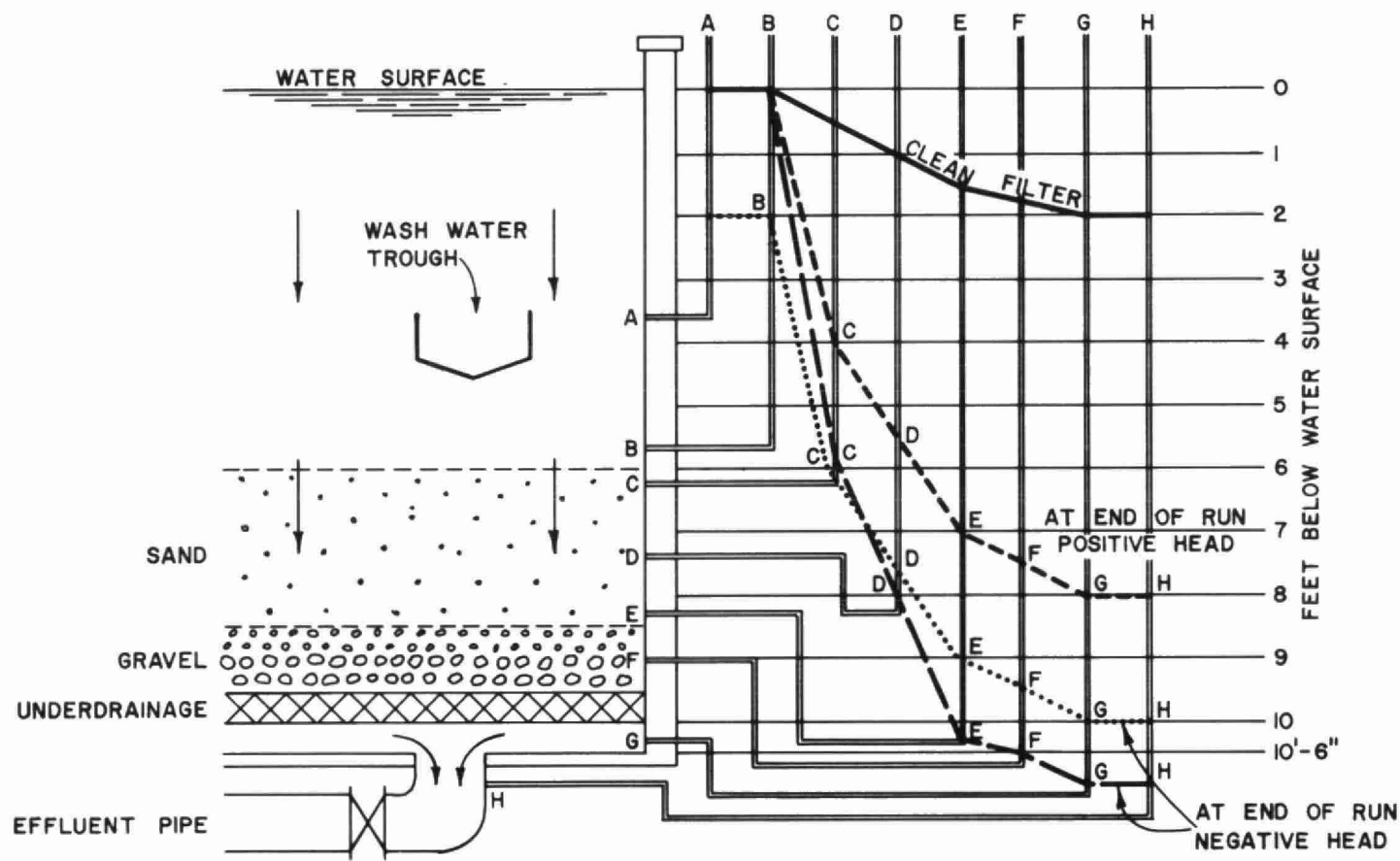


FIG. 5 - FILTER BED HYDRAULIC PROFILES (FILTERING)

GROUND WATER DEVELOPMENT

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INTRODUCTION

② No P// In the planning of any community, water plays a major part. Little progress can be made if water supplies are either inadequate or of unsatisfactory quality. As the community expands the water requirements in all phases of personal, communal, and industrial life becomes increasingly important. Water shortages can halt expansion with the result human welfare suffers accordingly. In Ontario alone, 35 percent of the population is dependent upon ground-water sources of supply. Ground-water development is therefore of continuing importance to a large section of the population.

TABLE

③
④ Before an attempt is made to discuss ground-water developments, it is essential to understand the natural environment and natural forces that are responsible for the occurrence, storage, and movement of ground water.

Many of the subjects and thoughts that will be used in this discussion have been referred to previously in the basic and intermediate courses; however, it is considered necessary to review and expand on some of them.

GROUND WATER OCCURENCE AND MOVEMENT

Ground Water Zones

⑤ The formations of the earth are divided into three major zones with regard to water supply:

1. Zone of Aeration
2. Zone of Saturation
3. Zone of Rock Flowage

The Zone of Aeration can be defined as the zone in which the open spaces in permeable formations are not filled regularly with water.

~~This zone can be subdivided into the following:~~

- A. ~~The Soil Water Horizon~~
- B. ~~The Intermediate or Vadose Water Horizon~~
- C. ~~The Fringe Water Horizon~~

This Zone does not contain water in usable quantities for water supply but acts rather as a medium through which moisture can travel downward to replenish the Zone of Saturation.

The Zone of Saturation occurs directly below the Zone of Aeration and is defined as the zone in which the functional permeable formations are saturated with water under pressure equal to or greater than atmospheric.

The Zone of Rock Flowage is defined as the deep part of the earth in which all rocks are under stresses exceeding their elastic limits. Water occurring in this zone is formed by chemical reactions within the rocks themselves and is of no value as a potential source of water supply.

The Hydrologic Cycle

Ground water is one phase of the hydrologic cycle. The cycle consists basically of precipitation, runoff (both on the surface and underground), transpiration, evaporation, and precipitation again. Ground water is an aspect of runoff which has resulted from the absorption of moisture at the earth's surface, downward movement of the moisture through the Zone of Aeration to the Zone of Saturation, and then lateral movement under the influence of gravity to a point of natural or artificial discharge. 6A

Geological Controls as Related to Ground Water

Geology affects the framework in which ground water occurs. It includes the stratigraphy and structure of the overburden and rock formations which together house the intricate ground-water system.

Geological factors relate chiefly to the formations and their water-bearing properties and hydrologic factors relate to the movement of water in the formations. The permeability,

or capacity of transmitting a fluid, of the formations is the result of the geological agencies. These agencies are responsible for the forming and altering of the overburden and rock strata thus developing their water-bearing properties.

The interstices in rocks and overburden deposits are the open spaces in which the water can occur and move. They differ widely in size, shape, arrangement, and aggregate volume. The openings through which water moves range in size from huge limestone caverns and lava tubes through all gradations to the minute pores in clay. Even smaller openings of molecular dimensions are of significance in relation to absorption phenomena. The interstices are generally irregular in shape, but different types of irregularities are characteristic of deposits of different kinds.

Interstices of the rocks and overburden materials can be grouped into two major classes. Original, or primary interstices that date back to the formation of the rock or overburden deposits and secondary interstices, such as joints, fissures, and solution passages which have developed later. Generally the original interstices have been altered by solution, cementation, recrystallization or other processes.

Water in unconsolidated materials moves through the interstices. Clean, coarse gravel is the most productive unconsolidated water-bearing material. Progressively poorer water-bearing materials range downward from clean, coarse sand to very fine sand and silt and heterogenous mixtures of fine and coarse particles such as are found in glacial till.

The consolidated rocks which were formed by solidification of molten rock and those developed from unconsolidated sedimentary deposits through pressure, cementation, and crystallization are commonly jointed and contain a large proportion of their water in these joint cracks. Less thoroughly cemented sandstones yield most of their water from pores and are amongst the most productive of the consolidated rocks. Other highly productive rocks include limestones and dolomites which contain water along joints, fractures and in solution cavities.

Rocks and unconsolidated sediments that do not yield water freely but which furnish water where better aquifers or water-bearing formations are lacking, include fine grained and poorly sorted overburden deposits and jointed crystalline rocks such as granite.

The capacity of any rock or overburden material to absorb, hold or yield water is dependent on its porosity which is the ratio of the aggregate volume of the interstices in a rock or soil to its total volume (This value is usually referred to as a percentage). In consolidated rocks the original porosity usually has been reduced by compaction or by deposition of cement in the pore spaces, although the total porosity may have been increased by development of fractures or by dissolving of some of the rock material.

Rocks of low porosity necessarily are limited in their capacity to absorb, hold or yield water but it is not necessarily true that rocks of high porosity are capable of yielding large quantities of water. One saturated rock may yield most of the water contained in its pores to wells or springs, whereas another having equal porosity but smaller pores, may retain practically all of its water and yield negligible quantities to wells. Where the porosity is identical, the difference is in the proportion of the contained water that is held by molecular attraction and the proportion that can be moved by gravity or hydrostatic pressure. Molecular attraction becomes increasingly significant with decreasing size of the interstices, because of the increased surface area of solid material to which water can adhere.

TABLE 8 → *A general* ———
The rate of movement of ground water is dependent on the capacity of the rock material for transmitting water. This capacity is known as the permeability. In permeable materials gravity is the force that moves water downwards from the land surface to the Zone of Saturation, thence laterally towards lower elevations and ultimately to places where the water is discharge by springs and seeps.

~~TABLE 8~~ In the development of ground water by means of wells, a hole is drilled into the Zone of Saturation and water drains from the overburden material or rock into the well. As the water is pumped, the reduced hydrostatic pressure at the well causes movement of water toward the well. The rate at which water can be withdrawn depends on the permeability of the material. Rock or overburden horizons that yield water in sufficient quantities to be of consequence as a source of supply are called aquifers or ground-water reservoirs. The ground-water reservoirs, like surface reservoirs, serve the same general purpose in that they tend to smooth out the daily and seasonal fluctuations in the amount of water supplied by precipitation. Ground-water reservoirs absorb water during periods of surplus and release it to springs or other avenues of discharge providing most of the stream flow during rainless periods.



Water Table and Artesian Aquifers

In areas where the materials are permeable down to the Zone of Saturation the aquifers are said to be under water-table conditions and the level of the water in the well and aquifers is called the water table. The water in this aquifer is unconfined.

Impermeable layers above the Zone of Saturation produce various complications to this simple picture. Such layers may be at the surface and practically all water from precipitation must collect on the surface or runoff over the land surface. Impermeable materials such as clay or glacial till may be present at various depths and retard downward percolation, so that there is an accumulation of water or temporary saturation immediately above the impermeable strata, this constitutes ground water that is perched above the main Zone of Saturation.

Many deep wells penetrate far into the Zone of Saturation and go through materials that range widely in permeability. As these wells are being drilled, water may rise markedly above the top of the aquifer that is being tapped and may even overflow at the surface. Such water is confined beneath less permeable material and is said to be under artesian conditions.

An artesian aquifer does not receive significant quantities of water by downward movement through the confining beds that overly it. Even in places where the confining bed is moderately permeable the artesian pressure opposes some of the downward movement. An artesian aquifer is replenished mainly in some area where the confining bed does not exist and where ground water is under water-table conditions. The area that supplied water to the artesian aquifer is known as a recharge area.

The amount of storage in an aquifer is no indication of the aquifers capabilities for sustained yield of water to wells or springs. The limit of perennial yield is set by the average annual recharge to the reservoir just as the useful yield of a surface reservoir is set by the inflow to it. Capability of the aquifer for sustained yield is determined by the quantity of inflow to that reservoir in its recharge area. This infiltration in recharge areas is of critical importance and the identification and delineation of recharge areas is as important as defining the position and extent of aquifers in an area.

Principal Occurrence of Overburden Aquifers

Unconsolidated deposits of sand and gravel generally form the aquifers that supply large quantities of ground water from the overburden. Other overburden deposits are also used as sources of ground water but usually these are capable of yielding little more than domestic supplies.

The types of occurrence of overburden aquifers may be grouped broadly as follows:

1. Glacial deposits
2. Buried valley
3. Watercourses

1. Glacial Deposits

Important ground-water reservoirs occur in permeable deposits left by melting ice associated with retreating glaciers. Eskers, kames, outwash plains, deltas, and sand dunes are all potential sources of ground water.

2. Buried Valleys

Buried valleys are no longer occupied by the streams that formed them. Drainage systems were disrupted by advancing and retreating ice during geological times. Permeable sediments left in channels were buried by younger sediments and often constitute favourable aquifers.

3. Watercourses

Watercourses are hydrologic units that involve both surface water and ground water. The wells tapping these aquifers frequently are so placed that water pumped from them is readily replaced by infiltration from rivers or streams. The water yield is generally large.

Principal Types of Bedrock Aquifers

In order to understand the water-bearing properties of the various rock types it is necessary to define the three classes into which all rocks are divided:

1. Sedimentary rocks are rocks formed by the accumulation of sediments in water or on land.

2. Igneous rocks are rocks formed by solidification from a molten or partially molten state.
3. Metamorphic rocks are rocks formed in response to pronounced changes of temperature, pressure, and chemical environment.

1. Sedimentary Rocks

In general sedimentary rocks are better aquifers than are either igneous or metamorphic rocks. The importance of sedimentary rocks as aquifers depends greatly upon the degree of sorting of the grains, the amount of cementation, and the type of cementing material. In order of importance they can be classified as follows: sandstone, conglomerate, dolomite, limestone, and shale. Water occurs within the interstices, joints and fractures.

2. Igneous Rocks

As a rule, igneous rocks are not good water producers. Some fine grained volcanic rocks are good aquifers due to permeable zones located in large joint openings and in zones of fragmentation that often occur between lava sheets. In the coarse grained igneous rocks water is confined to joints, cracks and along fault zones. These openings tend to become tighter and fewer in number with increasing depth.

3. Metamorphic Rocks

As in the case of igneous rocks, metamorphic rocks are generally poor aquifers, the water occurring mainly in joints, fractures and along fault and cleavage planes.

If the bedrock remained undisturbed from the time it was deposited or intruded, the study of ground water would be relatively simple. As it is, this is seldom the case as most rocks are subjected to varying amounts of stress which result in gentle warping or intense folding and fracturing. Deformation of this nature complicates matters. A water table may cut across a series of folded beds in such a manner that a particular geological horizon may in places lie above and in other places below the water table; movement along fracture planes may be such that an aquifer may abut against material that does not transmit water or a fracture plane may be filled with broken rock which could be important as a source of ground water.

An almost infinite number of situations may arise due to deformation which would necessitate a careful study of the rock structures before an appraisal could be made of the ground-water potential of the geological horizons under investigation.

GROUND WATER EXPLORATION

Modern ground-water development is the practical application of technical knowledge to the provision of water supplies for beneficial use, free from pollution and in accordance with the best principles of ground water conservation. This requires not only a knowledge of well-drilling equipment and methods, but also a comprehensive understanding of geology, hydrology and ground-water chemistry.

The larger ground-water users such as municipalities and industries require detailed information on the feasibility of proposed developments before actually undertaking any drilling. This information can be obtained at a reasonable cost by a well planned, preliminary investigation programme.

Prospecting and Testing

To establish definite and feasible ground-water developments, any drilling should be preceded by sufficient study and investigation. It is often necessary to resort to geological and geophysical methods in the field, using techniques that will obtain the required information quickly and with reasonable accuracy. From this preliminary work, it can be deduced where the best aquifers are located, what their extent and general nature is, and where test drilling is warranted.

1. Geophysical Exploration

Geophysical methods of exploration consist of the measurement and identification of changes in certain physical properties of the earth at or near the surface. At present, the geophysical methods most applicable to ground-water prospecting are the seismic, electrical-resistivity and gravity methods. They involve, respectively, the measurement of the rate of propagation of elastic waves in the earth, changes in the electrical resistance of the earth and the measurement of the total gravitational field of the earth. These three methods were

primarily developed for use in prospecting for petroleum and minerals. Adaptations and modifications of these basic methods are extensively used for ground-water prospecting.

When used in conjunction with geological data, the methods, under favourable conditions, give results which indicate the character and sequence of strata and the depth to bedrock. The full value of these methods are obtained in their correlation with, and as supplements to, other data.

2. Geological and Hydrologic Surveys

During a survey of this nature the exposed overburden and bedrock geology is carefully examined and hydrologic and water quality data are collected from existing wells, test wells, test holes and observation holes in the area under investigation. This information is compiled and plotted on a base map and used in conjunction with topographical details to construct geological sections and contour maps of the bedrock and water table or piezometric surface. (The piezometric surface is the level to which water from a given artesian aquifer will rise under full head). Details of the geology, the bedrock surface elevations and directions of natural ground-water flow, are used in association with pumping tests, water quality and geophysical data to determine the location and characteristics of the various aquifers in the area under investigation.

Test Drilling

If geological, geophysical and hydrologic conditions indicate that a further investigation of ground-water conditions is warranted, a test-drilling programme should be instigated. To put down test holes, standard drilling equipment should be used, the type depending on the nature and depth of the formations to be penetrated. As the purpose of all test holes is to obtain accurate formation data and representative samples, it is essential that the character of the holes and drilling methods used be such that the results are accurate and dependable. A programme of testing must be carefully planned in advance. The following procedures should be undertaken.

1. A sufficient number of test holes should be drilled to determine the area, thickness and depth of the best aquifer within the sector available for development.
2. Test holes should be at least 4 inches in diameter. Gauge holes can be 2 inches in diameter.

3. All formation samples should be as nearly representative of the natural formations as possible.
4. All formation samples to be examined should be placed in containers on which depth, location and other information is marked.
5. Samples of water from each aquifer penetrated should be taken during drilling.
6. All analyses of formation and water samples should be made by persons who are experienced in such work and are qualified to interpret the hydrologic, as well as the geological characteristics of the samples.

If the results of test drilling indicate that a favourable aquifer is present, a pumping test should be undertaken to establish the hydrologic characteristics of the aquifer.

Pumping Tests

A pumping test refers to a test carried out on a production or test well in which water is pumped at a constant rate or by means of a series of stepped, increased rates. Observations are made of the pumping rate, the duration of operation and the drawdown in the well and one or more nearby wells or gauge holes.

Report

On completion of the test-drilling programme all available data should be correlated and assembled in a written report and accompanied by a ground-water map.

GROUND WATER EXTRACTION

Well Design and Construction

Assuming that the preliminary investigation, prospecting and testing have been completed and the feasibility of developing a ground-water supply has been established, the next step is the construction of the necessary wells.

Selection of Well Type

One of the fundamentals of ground-water development is to select the proper type of well for the aquifer and water use.

The Committee of the American Water Works Association has prepared a complete set of well specifications. The specifications are as follows:

1. The proposed well should be constructed of materials that give the longest life under the soil, water, and pumping conditions which are already present in the ground formations or which may result from the operation of the well.
2. The proposed well should be constructed so that it will make the maximum quantity of water available for pumping with the least possible drawdown per gallon.
3. The proposed well should be so constructed that only usable water can enter it at any place and only through the openings provided for this purpose.

Testing for Capacity

Discharge

In all tests, the discharge of the pumped well should be maintained at one or more constant rate throughout the test period. If more than one rate is used, the change should be an abrupt step from one rate to another. If the step discharge test is used, the period of pumping for each step should be sufficient to provide the basis for extrapolating water level trends from each past period.

The measurement of discharge may be made by any suitable device that limits the error to less than five per cent. Prior to, during and after the test, the pumping from nearby wells which tap the formation being tested, should be stabilized if possible. If the regulation of pumping from outlying wells is not practical, records of their operation should be maintained.

Water Level

The observation of drawdown in the pumped well and observation wells should be made with a steel tape or electrical device. Readings should be taken to the nearest hundredth

of a foot during the period the well is pumped and also during recovery after the pump has been shutdown.

Prior to the performance of the test, each observation well should be tested to determine its sensitivity to water fluctuations within the aquifer. If the observation wells are not sensitive the condition should be rectified by further development.

The spacing of observation wells relative to the pumped well should be based on a knowledge of the geology, the transmission and storage properties of the aquifer to be tested, the duration of pumping and the location of nearby pumped wells.

Timing of Pumping Tests

The timing of water level measurements within the test period are of particular importance. The amount of water level decline at each observation well may be as great during the interval between the first and second minute after starting the test as during the interval from 1000 to 2000 minutes. The times of measurement should be very closely spaced at the start of the test or after each change in rate and should decrease in frequency as time progresses.

Analysis of Pumping Test Data

Properly conducted and interpreted, the pumping test provides the means of obtaining data on the ground-water hydrology within the area surrounding the well that is being tested. The information furnishes the basic data for the selection of well equipment. An economic study can be made of the relative importance of diameter, spacing and distribution of wells in any aquifer. The amount of interference, the effect on adjoining wells, and the long-range performance of the well can be predicted.

Ground Water Development and the Ontario Water Resources Commission

The Division of Water Resources of the Ontario Water Resources Commission is charged with the responsibility of collecting ground-water and related geological data in the Province of Ontario. This data is obtained largely from field

surveys, water-well records, observations wells and investigations. The Surveys and Project Branch of the division co-operates with municipalities and others interested in public water supplies.

At the request of municipal officials, the branch will undertake surveys to assess the ground-water supplies and make recommendations concerning improvements and additions to them.

A municipality may enter into an agreement to have the Commission finance and supervise a test-drilling or well-construction programme and the Commission will appoint a licensed well-drilling contractor to carry out the work under the direction of the Division of Water Resources. The Division is also actively engaged in the development of ground-water supplies for areas accepted as provincial projects.

Private individuals or corporations who require ground-water surveys of this nature are advised to contact consultants or well-drilling contractors who are familiar with this field.

Sanitary Construction of Wells and the Ontario Water Resources Commission

Under The Ontario Water Resources Commission Act it is required that wells be constructed in a sanitary manner. Individuals, firms or companies carrying on the business of boring or drilling wells for water must be licensed by the Commission. Regulations set out under the Act control the licensing of well drillers, the materials to be used in well construction, the sealing of openings in casings, the prevention of aquifer and well contamination and the sealing of wells containing non-potable ground water.

GROUND WATER CONSERVATION

15A Water conservation is equivalent to full use of water resources. In the past, insufficient thought and time has been devoted to the efficient use of ground water with the result that a number of areas are suffering from water shortages. A number of reasons for this situation are listed below:

1. Overconcentrations of industry and population.

2. Ground water withdrawal in excess of natural recharge.
3. Lack of artificial recharge.
4. Water wastage.
5. Location of communities or plant sites with inadequate planning.
6. Poor design of water consuming process equipment.

A number of the factors contributing to ground-water shortages result from lack of knowledge and unsound planning. On the other hand many industries and communities are located where circumstances beyond their control have caused water supply problems. It has been found that the long-term prediction of the demand for water is extremely uncertain. Nevertheless planning for the future is necessary.

By the study of water, by using operational measures for augmenting local supplies of ground water and by conservation, thorough efficient use and reuse can be developed. These procedures may not be sufficient in some circumstances and if this is the case, thought should be given to artificial recharge of ground-water reserves.



Artificial Ground-Water Recharge

Artificial recharge may be defined as the practice of increasing, by artificial means, the amount of water that enters ground-water reservoirs. The need for artificial recharge has occurred due to an increasing demand for fresh ground water. In a number of areas the withdrawal of ground water has exceeded natural recharge due to heavy well concentrations or low rates of natural recharge.

Some of the purposes for which artificial recharge is practised are as follows:

1. Supplement the quantity of ground water available.
2. Reduce or eliminate the decline in the water level of ground-water reservoirs.
3. Conserve and dispose of runoff and flood waters.
4. Store water to reduce costs of pumping and piping.

5. Reduce, prevent, or correct salt water intrusion.
6. Store clear, cool water in winter for use during the summer.
7. Allow heat exchange by diffusion through the ground.

Methods of Recharge

Water can be supplied to aquifers by means of recharge wells and natural and artificial recharge ponds. In the case of the recharge wells, water is allowed to flow by gravity or is pumped directly to the aquifer by means of a well or well system. Natural and artificially constructed recharge ponds allow surface water to accumulate and percolate downward through the ground to supplement the ground-water supplies.

Water that is to be used for artificial ground-water recharge may require treatment. The amount of treatment needed before the water can be allowed to enter the ground depends on the quality of the surface water to be recharged and the distance from the point of recharge to the nearest producing well.

~~As the water from recharge ponds percolates downward it is cooled by the earth. In a case of recharge from a pond in Kalamazoo, Michigan, the downward velocity was measured at 1.75 feet per day and the cooling rate was 1°F for each 19 feet of travel through the earth. These facts when established, make it possible to estimate the distance a pumped well should be located from the source of recharge in order not to affect the well water temperatures.~~

Artificial recharge, where applicable, serves several valuable purposes that aid in solving many water supply problems. Among the most important of these are the storing of runoff and flood waters for use when needed during dry seasons, the reducing or eliminating of effects of over-development of ground water and the supplementing of existing ground-water supplies.

Water Conservation and the Ontario Water Resources Commission

Part of the role played by the Ontario Water Resources Commission in the controlling and conservation of water in the Province of Ontario is set out in Section 16 (1) (a) of The Ontario Water Resources Commission Act, which is as follows:

"Notwithstanding any other Act, it is the function of the Commission and it has the power to control and regulate the collection, production, treatment, storage, transmission, distribution and use of water for public purposes."

The control with regard to ground-water conservation is extended through Section 28 (a) of The Ontario Water Resources Commission Act which regulates and requires a permit for the taking of more than a total of 10,000 gallons of water in a day by means of a well, excavation, inlet, or structure or any number of combinations of these works where such works have been constructed or installed after the section came into force. For example, a permit is required for the taking of water for irrigation, municipal, commercial, and industrial use; for the construction and operation of quarries and gravel pits and for the diversion or storage of water. A permit is not required for and there is no regulation of the taking of water for domestic and farm use and for fire-fighting purposes.

By means of this legislation it is possible to keep records of the larger water users and to regulate the amount of ground water taken from any one area. The legislation should lead to an equitable sharing of water amongst consumers and the protection of one of the Province's most valuable natural resources.

CONCLUSION

17 There has been a pronounced increase in per capita consumption of water in recent years and this trend will probably continue in future. In order to prevent economic losses that result from overdevelopment of ground-water resources, the safe perennial yield of aquifers should be determined as accurately as possible prior to the maximum development of these resources. The development of water supplies must be maintained in balance with replenishment.

INDUSTRIAL WASTES AS SOURCES OF
TASTE AND ODOURS

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INTRODUCTION

In this lecture, the contamination of raw water supplies by industrial wastes will be considered from the standpoint of how and to what degree industrial wastes affect the taste and odour of water and what measures can be taken to correct the problem.

DEFINITION OF THE PROBLEM

Biologically speaking, the sensations of taste and odour are similar. Both the tongue and the nose contribute to the discernment of the tastes of foods and drinks, with the tongue able to identify sweet, sour, bitter or salty qualities, while the nose identifies flavours. The sensation of taste and odour is relative, that is, there is a greater ability to discern differences in taste and odour and we have probably evolved this ability as a protection against disease and food poisoning.

One can become accustomed or acclimatized to the taste and odour of many substances, such that their absence is quickly noticed. For this reason, it is conceivable that taste and odour problems could be attributed to the absence as well as the presence of certain substances in potable waters. For example, distilled water, which is virtually pure water, has a flat unpleasant taste; we notice the absence of minerals usually present in water. However, in this discussion, I will consider only the presence, rather than the absence, of foreign substances as taste and odour producers.

You may be familiar with the noticeable, and sometimes unpleasant, difference in taste between municipal water supplies in Ontario. This is particularly noticeable between municipalities having surface water supplies and those having ground water sources of supply, and, in most cases, this difference can be attributed to differences in mineral content. Almost all cases of taste and odour problems can, however, usually be attributed to compounds containing carbon and hydrogen and traces of nitrogen, sulphur or

phosphorus. These compounds are called organic materials and are often of animal and vegetable origin.

It would be convenient if I could give you a list of taste and odour-producing substances with a corresponding table of actions to be taken to counteract their effects. However, in attempting to do this, it becomes apparent that almost anything soluble in water can impart a taste and odour to water. This is further complicated by the fact that for practical purposes almost anything is soluble in water to some degree (we are literally washing ourselves away in taking a bath).

Also, the sensation of taste and odour is extremely sensitive, such that it is possible to notice the presence of some compounds in concentrations which cannot be detected or isolated by many very sensitive laboratory instruments. The fact that phenols have been given so much attention in taste and odour problems is probably because it is comparatively easy to detect and analyse for this chemical. For these reasons, a relatively small number of compounds have been identified as taste and odour producers.

The problem is further complicated by the fact that tastes and odours in water may be additive, that is, the combined effect of some taste and odour-producing substances may be much worse than the individual substances. For example, the taste of phenol in water becomes very noticeable after the water is chlorinated.

NATURAL SOURCES OF TASTES AND ODOURS

Before dealing specifically with industrial sources of tastes and odours, I should like to mention other possible sources. It has been estimated that at least 50% of all taste and odour problems can be related to natural phenomena. You are probably familiar with the seasonal nature of the incidences of some taste and odour problems, particularly where the sources of supply is a lake or river. These are usually related to changes in water quality brought about by changes in climatic conditions. The most obvious examples of this are run-off from melting snow and seasonal rainfall, both of which tend to flush decayed animal or vegetable matter into lakes and rivers. The other factor is the turnover in lakes and rivers caused by the formation or melting of ice cover. When ice melts, the resulting water, being more dense, sinks to the bottom causing a turnover of a body of water

such that bottom sediments are stirred up and may give rise to taste and odour problems from the presence of decayed animal and vegetable matter in the bottom sediments. A similar effect may also occur in the winter, as ice starts to form.

Seasonal variations in ground water quality may also occur due to fluctuating water levels, but these are more likely to be associated with excessive mineral content rather than the presence of materials of animal or vegetable origin. In this category, there are excessive amounts of salt, iron, hardness and sulphides. These materials may occur naturally in the soil, and variations in ground water levels may influence the degree to which they become dissolved in the ground water.

INDUSTRIAL SOURCES

Industrial water consumption in Ontario probably exceeds one billion gallons per day. It takes 25 gallons of water to produce a gallon of beer, 8,000 gallons are used in the manufacture of an automobile and up to 1,000 gallons may be used to produce a full tank of gasoline to run it.

Almost all of the water used in industry is discharged directly or indirectly to natural waters in the Province, which are often also the source of municipal water supplies. It is understandable, therefore, that industrial wastes are sometimes sources of taste and odours in potable waters.

Industries can be broadly classified under three main headings, according to the type of wastes discharged from the plants, namely: Organic, Chemical or Inorganic.

Organic Classification

Industries in this classification are largely those involved in the processing of animal and vegetable raw materials. The following types of industries are included in this category:

Meat Packers	-	Abattoirs, poultry plants and rendering plants
Tanneries		
Milk Plants	-	Dairies, creameries, cheese factories, condensereries and dried milk plants
Vegetable Packers		

Starch Plants and Potato Chip Manufacturers

Breweries, Distilleries and Wineries

Pulp and Paper Mills

Textile Mills

All of these plants produce wastes which are highly organic and, consequently, tend to exhaust the dissolved oxygen content of the receiving stream, as they decompose. This is an approximate definition of pollution. However, in dealing with the taste and odour properties of these wastes, some consideration must be given to more detailed waste characteristics.

Wastes from the food processing industries may have the taste and odour of the food being processed. In addition, other tastes and odours likely to occur may be described as musty, fishy, sour or salty, and are usually the result of the decomposition of nitrogen compounds present in many foods.

Tastes and odours likely to be associated with wastes from pulp and paper mills may vary from sulphide and turpentine odours to phenolic medicinal tastes and from tannery wastes, the taste and odour may be sulphide, fishy, or musty in nature. Textile wastes may contribute soap or detergent tastes to water, as well as alkaline or earthy tastes.

Chemical Classification

Industries in this classification are essentially those engaged in the manufacture of chemical products from simple raw materials. Many of the industries in this group are extensions of the petroleum and petrochemical industries, which are also included. The remainder consist of heavy chemical industries manufacturing acids, alkalis, fertilizers, etc. from mineral raw materials.

The wastewaters from the petroleum and petrochemical group industries may have a phenolic, medicinal, sulphide, soapy or oily taste, while the wastes from the heavy chemical group may have an acidic, salty or earthy taste.

The following industries are included in this classification:

Oil Refineries

Petrochemical Plants manufacturing:

detergents
insecticides
pesticides
solvents
plastics

Drug and Pharmaceutical manufactures

Fertilizer Plants

Heavy Chemical Plants manufacturing:

acids
alkalis
industrial gases
chlorine

Inorganic Classification

Industries included in this category are a diverse group of manufacturing plants, mainly engaged in the processing or manufacture of metal or mineral products. Included are the following industries:

Steel Mills
Mining, Smelting and Metal Refining
Metal Working and Manufacturing
Plating and Metal Finishing
Ceramic Industries - glass, brick and tiles

The plating, metal working and manufacturing industries are sources of mineral acids and metal finishing solutions, as are steel mills, and these wastes are potential sources of acidic and metallic tastes. Steel mills are also sources of phenolic and other chemical tastes from coal tar, produced in the manufacture of blast furnace coke.

Ceramics and mining industry wastes are mainly contaminated with inert suspended solids, which are unlikely to present taste and odour problems, although certain plants may be sources of acidic or metallic tastes.

CORRECTION OF THE PROBLEM

It is important to remind you that all of the previous discussions give a very general description of the possible effects of certain industrial wastes on taste and odour. We have now reached the stage where it is possible to recognize, in a general way, the possible sources of taste and odour problems when they occur. However, by this time of course, it is often too late, the taste and odour has probably entered the distribution system so you can now expect calls from irate consumers.

In some cases, it may be possible to transfer to another source of supply. However, it is doubtful that an alternative supply will be available to most operations. Therefore, an attempt must be made to control the problem with the treatment equipment available at the plant. Previous courses have outlined the measures that can be taken at the filtration plant. Activated carbon treatment appears to be the best all round cure for most taste and odour problems. Chlorine dioxide is effective for most oxidizable taste and odour producers, such as phenols. Aeration or free residual chlorination, before or after filtration, appears to be effective against sulphides and iron.

Taste and odour problems of a natural origin, whether seasonal or continuous, probably occur frequently enough that their control can be at best a routine or at worse a painful necessity. However, problems due to industrial wastes can and do occur unexpectedly, and often at most inconvenient times. Continuing problems from industrial wastes usually receive the prompt attention of the OWRC, Division of Industrial Wastes, and with a little prodding of the offending industry we can, hopefully, achieve a solution.

Unexpected batch discharges of taste and odour-producing industrial wastes are the most difficult problems to correct. These may be the result of spills or accidental losses at an industry, and it is rarely possible to trace them back to the offending industry without some prior knowledge of the spill.

About the only way this can be controlled is by a sustained public relations programme to persuade industry to report all accidental spills.

It is, therefore, clear that some attempt must be made to anticipate problems, working on the assumption that if something can go wrong, it eventually will. The following is a short list of industries which, from experience, are known to be the principal industrial sources of taste and odour-producing wastes:

Oil Refineries

Petrochemical Plants - Synthetic Textiles
Polymers
Dyestuffs
Insecticides
Detergents

Steel Plants - (coking operations)

Pulp Mills

Tanneries

Food Processing Plants- Canneries
Meat Packers
Milk Plants

All of these industries practise one or more of the following methods of disposal:

1. Disposal to a lake or river (with or without treatment).
2. Disposal to a municipal sewer (with or without treatment).
3. Spray irrigation.
4. Deep well disposal.

Methods (1) and (2) will affect the quality of surface waters, while (3) and (4) are potential sources of ground water contamination.

Most of these industries are generally of the primary type. That is, in most cases, they are manufacturing products which may be used in other enterprises. It is, therefore, worth bearing in mind, for example, that the absence of an oil refinery in your area does not necessarily eliminate the possibility of tastes and odours due to oil or phenols, since there is likely to be at least one gas station.

REQUIREMENTS FOR WATER WORKS EXPANSION

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INTRODUCTION

The demand for water in every municipality is increasing so fast in this modern age that it is often difficult for municipalities to keep up with it. For the past several years, the per capita water usage has been considered by design engineers as 100 gallons per day, but in many cases this figure appears to be unrealistic. Each year the per capita usage is increased with more and more installations of automatic dish and clothes washers, swimming pools, air conditioning units, etc. In some municipalities the per capita usage is approaching 120 gallons per day, and in another twenty years this figure might rise to as high as 150 gallons per day.

This means that most of the water works in the province either immediately, or in the very near future, will require expansion.

PURPOSE

The purpose of this paper is to outline the methods of assessing the various components of a water works system with respect to their reliability and capacity and to determine when in the future, replacement and/or additions will probably be required.

WHEN IS A WATER WORKS SYSTEM ADEQUATE

Before entering into any discussion as to when a water works system requires expansion, it would be well to have a definition of what constitutes an adequate system. For the purposes of this paper the definition used by the Canadian Underwriters Association, which sets the standards for municipal fire protection will be used. According to the C.U.A. a water works system is considered to be adequate if it can deliver fire flow for specified periods of time with consumption at a maximum rate. The requirements for fire flow vary with population and these

requirements are shown in Table I of the appendix. If delivery of the amounts of water specified in this Table is also possible in certain emergency or unusual conditions the system is considered to be reliable.

In order to provide reliability, duplication of some or all parts of the system are required, the need for duplication being dependent upon the extent to which the various parts may reasonably be expected to be out of service as a result of an emergency or unusual conditions. The provision of storage, either as part of the supply works or on the distribution system, may partially or completely offset the need for duplicating various parts of the system, the value of the storage depending upon its amount and location.

ROLE OF THE WATER WORKS OPERATOR IN PLANT EXPANSION

The water works operator is a very important link in the chain of persons charged with the responsibility for treatment and delivery of water to consumers and in any expansion of the associated treatment plant. The operator's role in the treatment and delivery of water is obvious. His role in plant expansion while of considerable importance is not as apparent at first glance.

Studies pertaining to any expansion rely heavily on the analyses of past data. The water works operator is the person most intimately associated with the physical treatment and delivery of the water to the consumers. His knowledge and experiences therefore can have a significant bearing on any expansion design. By ensuring that careful records are kept of daily operating procedures and the problems involved, he can have a significant affect on future design.

Too often one finds that records at plants are sketchy and incomplete because in many municipalities the keeping of good records is viewed as an expensive and unproductive function. The value of good records is apparent only when problems arise or when expansion is contemplated and it becomes necessary to search the records to see what has been and is happening. One cannot stress too strongly the need for keeping the best possible records that time of the operating personnel and budget permits. For it is only by a thorough study of the background of a water system that a reasonable assumption of future requirements can be made.

RECORDS

The following list gives examples of the types of records useful in evaluating a system and its requirements. It should be remembered that these are only a general outline, each operator can supplement these based on the specific operations of his plant.

1. A continuous record of the number of customers served;
2. Water consumption by months, from customer meter records;
3. Records of the capacity of wells, streams, treatment units, and pumping plants;
4. Records of actual output of wells, streams, treatment plants, and pumps;
5. Records showing the time of operation of intermittently operated facilities;
6. Records of flow requirements for maximum month, day and hour;
7. A map of the distribution system on which leaks and repairs are plotted;
8. Pressure charts for strategic points, including all pertinent data relating to such items as draft and operation of pumps at the time the charts were made;
9. A map of the distribution system showing proposed long-range enlargement and expansion plans.

From such data the engineer may determine future annual and maximum short period water demands, size and capacity of current and future plant facilities, weak points in the distribution system, adequacy of local storage and pumping facilities, and other operating and construction design factors. In communities where the present water supply is limited, special records should also be kept to establish the time when supplemental supplies will be needed. Such records would include not only studies of present supplies but also data on stream flow, water quality, ground water levels, and recharge rates of potential supplies.

SOURCE OF SUPPLY

The quantity of water at the source should be adequate to supply the total water demand of a community from that source, as well as a reasonable surplus for anticipated growth, with due

regard to the region within which the supply must be developed. It is good sanitary engineering practice to have a supply, the total available yield of which during a drought period should be greater than the estimated daily demand 10 years in the future. It should be noted that during a drought period when a surface water supply may be substantially reduced and the ground water table for a well is likely lowered, the demand for water is much greater.

For surface water supplies the water level and for well supplies the static level, drawdown, and recovery should be regularly measured and recorded. Records of unusual conditions are also useful. For example water quality and levels during particularly severe storms or floods can have a bearing on the type of treatment capabilities utilized in the future.

INTAKE

With respect to intake capacity, facilities should be provided for:

- (1) consumer requirements
- (2) in-plant process needs (e.g. filter backwash water)
- (3) fire flow requirements.

As far as fire flow requirements are concerned, any one of the following three conditions may exist:

- (a) If no storage is available following intake, the intake capacity must be large enough to meet fire flow requirements, customer demand and in-plant process needs.
- (b) If storage facilities following intake facilities are of sufficient capacity to meet all fire flow requirements, the intake capacity must be large enough to supply only in-plant and customer needs.
- (c) If storage facilities following the intake will supply less than fire flow requirements, the intake facilities must be sized accordingly.

To assess the intake capacity the following questions must be answered.

1. What is the nominal, minimum daily storage capacity at the plant and on the distribution system?
 2. During the past 5 years:
 - (a) What was the maximum amount of water delivered to the distribution system in a day
 - (b) What was the maximum amount used a day for in-plant process
 - (c) What is the required fire flow now (See Table I)
 - (d) What flow rate can be delivered for required fire flow duration (#1 x Storage factor from Table I)
- .°. Required intake capacity = a + b + c - d.

The above figure should be compared with the actual capacity of the intake. It is advisable that the maximum intake capacity be checked if possible by increasing the low lift pumping rate to a point where the water level in the pump well just begins to drop.

Preferably, there should be capacity in the intake facilities to meet the estimated demand five years in the future keeping in mind that the link with the least capacity fixes the overall capacity.

TREATMENT UNITS

The required capacity of the treatment units can be determined in a similar manner as was the intake with the exception that the total storage used should be only that which is available following the treatment units.

The operator should then be aware of which of the following unit processes has the least capacity.

1. Flocculation basins;
2. Sedimentation basins;
3. Sand, anthrafil or diatomite filters;
4. Iron and/or manganese removal units; or
5. Ion exchange softeners.

The least of these should then be compared with the required capacity.

In addition to the treatment units, the chemical feeding equipment capacities should be thoroughly evaluated. There have been many instances where this equipment has been satisfactory for average conditions only and was not suitable for sudden changes in raw water quality. For example, several of the water supplies between Metropolitan Toronto and Burlington are equipped with chlorine dioxide facilities to remove chlorophenol tastes in the water. On occasion, there have been accidents at nearby refineries, oil dumpings from freighters, industrial and other waste discharges which have resulted in high phenol concentrations that could not be adequately treated due to limitations in feeding equipment.

The settings on chemical feed machines during normal conditions should be at approximately mid-scale and no more. Therefore if the quality of the raw water deteriorates and more chemicals are required, a rapid adjustment may be made without exceeding the capacity of the machine. If chemicals are being fed at your plant close to the maximum rate afforded by the feeder and if the dosage is subject to frequent variations it may be necessary to have the machine altered for a higher feed rate range or replaced altogether.

It is noted that for all feeders necessary to proper treatment, (e.g. chlorinators) duplicate units should be provided.

PUMPING EQUIPMENT

Pumping capacity, where a system is supplied by pumps, should be sufficient, with the two most important pumps out of service, to maintain the maximum daily consumption rate plus required fire flow at required pressure.

The pumping system, together with the storage on the distribution system, should be able to supply the peak hourly demand for a number of hours that varies with the size of the municipality (see Table II appended). This also applies to the pumping facilities in booster pumping stations. It is desirable to have standby power sources available for the pumps.

STORAGE FACILITIES

If a water system is without a storage reserve in a form that can provide the system with adequate water at the proper pressure, it is necessary to provide more pumping and distribution facilities. In any water system, the withdrawal of water varies greatly, and without some form of elevated storage it is necessary to have pumping equipment of varied capacities in order to carry fluctuating loads. This can be accomplished by several pumps of different capacities or variable - speed pumps whose operation is controlled by main pressure. The latter method usually requires expenditures for mains and pumping equipment and results in an uneconomical pumping system, because the pumps do not operate at their greatest efficiency for the larger part of the time.

The introduction of storage of proper volume, location, and elevation results in the following advantages:

1. Reduces size of purification facilities or number of wells required;
2. Reduces necessary capacity of high lift pumping equipment;
3. Reduces size of transmission mains;
4. Makes uniform pumping rates possible;
5. Reduces friction head losses;
6. Provides uniform water pressure;
7. Maintains service during interruptions in power supply or failure of equipment;
8. Reduces operating costs by making it possible to operate pumping equipment at better efficiency and lower pressure. This is particularly true with motor operated pumps;
9. Provides water for fire protection. Water already available in storage is more certain than water which must be pumped;
10. Permits pumping to continue at a uniform rate during off peak consumption in refilling storage.

There are other uses of storage facilities other than to equalize flow, provide water during peak demand periods and for emergency reserve. A reservoir may be used as a surge tank on a long supply main from a pumping station. A small elevated tank may be connected to the raw water line near a river or lake pumping station to absorb line surges and ensure smooth and economical operation of the plant. Elevated storage may also be used frequently for an auxiliary water supply for plant operations. A tank designed with a low ratio of depth to diameter might be provided for filter backwashing at a uniform rate.

Example:

To illustrate the advantage of storage facilities, particularly to meet fire fighting requirements, it is assumed that the underwriters require a withdrawal rate from the hydrants of 1250 GPM for a period of 6 hours in addition to the regular consumption during that period. If this is equal to a total requirement for the 6 hour of 500,000 gallons, and if the supply is from wells, the capacity of the wells and pumping equipment must be sufficient to deliver 500,000 gallons in 6 hours, or 1,390 GPM. In such a community the average consumption rate probably would not exceed 140 gpm; therefore, without any storage, the wells and pumping equipment would have to be capable of delivering 1250 GPM for fire protection only. Pumping at that rate would be rather infrequent throughout a year, but it would represent a considerable material investment and operating expense.

If this same municipality had 100,000 gallons in elevated storage, 400,000 gallons in ground storage, a well with a dependable supply of 300 GPM and two 300 GPM booster pumps, during the 6-hour period the well supply would be equal to 108,000 gallons leaving 392,000 gallons to be supplied from elevated and ground storage. Under these conditions, at the end of 6 hours, neither the elevated storage nor the ground storage would be depleted and there would be a good supply for fire protection. The deep well pump and booster pumps would be sufficient to pump the normal requirement and a booster would be available for standby service. As a protection against a possible breakdown in the deep well turbine pump, however, this community should also have a fully equipped well for standby service.

DISTRIBUTION SYSTEM

The standards outlined for distribution systems by the Canadian Underwriters Association are generally as follows:

Pipes

Smallest pipes in gridiron	6 - inch
Smallest branching pipes	8 - inch
Largest spacing of 6-in. grid (8-in. used beyond this value)	600 - feet
Smallest pipes in high value district	8 - inch
Smallest pipes on principal streets in central district	12 - inch
Largest spacing of supply mains or feeders	3000 - feet

Gate Valves

Largest spacing on long branches	800 - feet
Largest spacing in high value district	500 - feet
Suggested spacing in single main systems	(1 on each main at intervals of 2 blocks; 1 at the junction of the service header with its main and one on each of the 2 branches to the hydrants)

Hydrants

Areas protected by hydrants	(See Table III)
Largest spacing when fire flow exceeds 4,200 gpm	200 - feet
Largest spacing when fire flow is as low as 1,000 gpm	300 - feet

SUMMARY

It is extremely important that an operator be fully aware of the capacities of the various components of his water works system and to what extent the actual hydraulic loading is approaching the design figures under average, emergency or unusual conditions. Inspections with required maintenance should be performed regularly. Detailed records should be kept of all phases of operations. Good records are the key to any evaluation of an existing system and requirements for future expansion.

The foregoing has been an attempt to outline some of the parameters utilized in evaluation and design of existing and future facilities. It is hoped that by providing you with some insight into the methods used in evaluating the need for plant expansion that you will be in a better position to provide the type of information required and thus perform a real service for your municipality.

APPENDIX

TABLE I
REQUIRED FIRE FLOW

<u>Population</u>	Required Fire Flow for Average Municipality		<u>Duration</u> (Hours)	<u>Storage</u> Factor
	<u>(GPM)</u>	<u>(MGD)</u>		
1,000	840	1.21	4	6.0
1,500	1050	1.50	5	4.8
2,000	1250	1.80	6	4.0
3,000	1450	2.09	7	3.4
4,000	1650	2.39	8	3.0
5,000	1900	2.74	9	2.7
6,000	2100	3.02	10	2.4
10,000	2500	3.60	10	2.4
13,000	2900	4.18	10	2.4
17,000	3300	4.75	10	2.4
22,000	3700	5.33	10	2.4
27,000	4200	6.05	10	2.4
33,000	4600	6.62	10	2.4
40,000	5000	7.20	10	2.4
55,000	5800	8.35	10	2.4
75,000	6600	9.50	10	2.4
95,000	7500	10.80	10	2.4
120,000	8400	12.10	10	2.4
150,000	9200	13.25	10	2.4
200,000	10,000	14.40	10	2.4

Over 200,000 population, 10,000 GPM, with 1700 to 6600 GPM additional for a second fire, for a 10-hour duration.

The previous table is the standard adopted by the C.U.A. for the required fire flow in the high value or principal mercantile district of average municipalities. This table is based on the formula:

$$G = 850\sqrt{P} (1 - 0.01\sqrt{P})$$

where G = required fire flow in gpm
and P = population in thousands

One third of the required fire flow determined by the above formula is considered to be the probable loss of water from broken connections, abandoned flowing hydrants, and other causes incident to a large fire in a high value district.

The C.U.A. do not ordinarily recognize any water works system incapable of delivering 200 GPM for two hours for fire protection. To overcome friction loss in the hydrant branch, hydrant, and suction hose, a minimum residual water pressure of 20 psi is required during flow. In special cases a minimum pressure of 10 psi is permissible.

TABLE IIREQUIRED DURATION FOR FIRE FLOW

Required Fire Flow (GPM)	Required Duration (Hours)
less than 1050	4
1050 and greater, but less than 1250	5
1250 and greater, but less than 1450	6
1450 and greater, but less than 1650	7
1650 and greater, but less than 1900	8
1900 and greater, but less than 2100	9
2100 and greater	10

TABLE IIISTANDARD HYDRANT DISTRIBUTION

Fire Flow Required (GPM)	Average Area per Hydrant (square feet)
850 or less	120,000
1650	110,000
2500	100,000
3300	90,000
4200	85,000
5000	80,000
5800	70,000
6600	60,000
7500	55,000
8400	48,000
9200	43,000
10,000	40,000

Source: Canadian Underwriters Association

SPECIFIC APPLICATION OF CHLORINE

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GENERAL

The lectures given on chlorination at the first two courses covered the purpose of chlorination, chlorine chemistry, bacteriacidal effectiveness, chlorine residual determination, types of chlorine residual, equipment, routine operation, emergency operation, and adverse bacteriological results.

The coverage given specific applications cannot, because of time, be complete. However, with the study of a number of examples it is hoped that the operator will be more capable of evaluating and controlling the chlorination program that may be under his supervision.

The lecture will cover such items as:

- (a) the point of application;
- (b) the point of testing for chlorine residuals;
- and (c) the sequence of application of various types of chemicals involved in a chlorination program.

To complete the picture the types of treatment to be applied are also covered. At times, the sequence of treatment will be shown on a type of chlorination program which may not be the most effective for a specific requirement. Nevertheless, full coverage is given because equipment limitations, finances, or emergency may prevent the use of the most effective method of treatment.

TYPE OF TREATMENT PLANTS

For simplicity the various chlorination practices will be applied to the following types of water plants:

1. Surface Water Supplies

- (a) Complete treatment plant including
intake well,
coagulation basin,
settling basin, (or solid contact unit),
filters, and
clear water storage.
- (b) Pressure filter plant without pretreatment and
including
intake pump well, and
filters.
- (c) No treatment
intake well.

2. Ground Water Supplies

- (a) Simple well supply
- (b) H₂S removal including
aeration, and
retention facilities.
- (c) Iron removal including some or all of the
following
aeration,
filtration, and
clear water storage.

TYPE OF TREATMENT

Where applicable, the following types of treatment will be considered:

- (a) Marginal chlorination for disinfection;

- (b) Free residual chlorination for disinfection and taste and odour control;
- (c) Combined residual by intent for maintenance of residual in a distribution system and/or taste and odour control;
- (d) Chlorine dioxide for taste and odour control.

SPECIAL INTENT

In setting up a chlorination program, every effort should be made to provide as long a contact period as possible and a free available chlorine residual. Where short contact periods are encountered, higher residuals are recommended. It is also suggested that pre and post chlorination be practised wherever possible.

COMPLETE TREATMENT PLANT

(a) Marginal Chlorination

Marginal chlorination is chosen where the water quality is good and taste and odour problems are not encountered. Pre and post chlorination is suggested with the application points being to the intake well or pump suctions of the low and high lift units. A minimum residual of 0.1 ppm is maintained on the filter effluent. The plant effluent must have 0.2 ppm chlorine residual.

(b) Free Residual Chlorination

Free residual chlorination may be required part or full time for the control of chlorophenolic tastes and to ensure the disinfection of a poor quality water. The numbers of possibilities involved in effecting a free residual chlorination program are many, but generally, the total chlorine dose is applied in the low lift well or pump suction with dechlorination being effected on the plant effluent. The residuals required will vary greatly, but a free residual of 2.0 ppm might be called for on a low demand water, while higher residuals may be required on an organically rich water. Dechlorination is effected with sulphur dioxide or ammonia, to lower the residual below a taste causing level and not below 0.2 ppm. The final free available

chlorine residual could be converted to a combined residual to prevent chlorinous tastes developing in the distribution system or to maintain a residual throughout the distribution system. The combined residual can be created with the addition of ammonia.

(c) Combined Residual Chlorination

To prevent chlorophenolic tastes a less active chloramine residual can be used for disinfection purposes. This is not the most effective method of control, but it may be satisfactory in certain instances. In effecting this type of program, the ammonia is added before the chlorine and its dosage must be varied to allow for the natural ammonia content of the water. In some instances, the natural ammonia content may be sufficient to create a completely combined residual. As a chloramine residual is less effective as a disinfectant than a free residual, the residual should be maintained at a higher level throughout the plant. If at any time the filters appear to have a high chlorine demand, special measures may be required to destroy nitrifying bacteria.

When the intent of the combined residual program is to subtain a chlorine residual in the distribution system, the ammonia may be added at the post position in the plant.

The residuals that are required in effecting a combined residual program may vary greatly, but a trace of 0.1 ppm on the filter effluent and 0.2 ppm or more on the plant effluent can be considered as minimum. The residual on the plant effluent may have to be as high as 0.5 ppm to be effective.

(d) Chlorine Dioxide

Chlorine dioxide treatment is most effective for the control of chlorophenolic tastes. For a complete treatment plant the chlorine demand should be satisfied with the more economical chlorine with the chlorine dioxide being applied at the post position. The residuals maintained are similar to that specified for marginal chlorination. When taste problems persist the post residual is increased. The usual proportion of chlorine to sodium chlorite is 1:1, but the sodium chlorite feed may be increased to provide a ratio of 1:2 in an emergency. In very difficult instances, chlorine dioxide is used at both the pre and post positions.

PRESSURE FILTERS WITHOUT PRETREATMENT

(a) Marginal Chlorination

Where the water quality is good and taste problems are not encountered marginal chlorination may be chosen. The pre chlorination application point could be a pump well, which may double as a settling tank, otherwise the pump suction is used. If more or less continuous operation is assured or if the intake well is small the intake or pump well application point is to be preferred.

The minimum chlorine residual on the filter effluent before the post chlorination application point is 0.1 ppm. If near continuous operation is assured, post chlorination may not be necessary. For either method of operation the plant effluent must have a minimum residual of 0.5+ ppm unless a two hour contact period is assured in which case the minimum residual is 0.2 ppm.

In a pressure filter operation it is often difficult to accurately control the chlorine residual in the plant effluent. Therefore, a higher than normal chlorine residual may be specified. A higher residual will also assist in overcoming a shorter in plant retention period.

(b) Free Residual Chlorination

Because of shorter retention periods and the lack of in plant storage, it is often difficult to effect a free residual chlorination program in the pressure filter plant specified here. In cases where the organic and ammonia content in the raw water is very low a free residual chlorination program may be possible without dechlorination.

(c) Combined Residual Chlorination

The combined residual program for the pressure filter installation is created for the same reasons and in the same way as outlined for the complete treatment plant. The one exception is the difficulty encountered in maintaining steady operating conditions. The lack of clear water storage facilities greatly reduces flexibility. If taste control is not involved post application of ammonia is to be preferred. Where water quality conditions are not consistent, the ammonia level of the raw water must be continually determined so that

the required ammonia dosage may be accurately determined. Excess ammonia will act as an enrichment media for growths in the distribution system, therefore, care should be taken to ensure that only the minimum amount is used.

(d) Chlorine Dioxide

The detail given for the complete treatment plant can be duplicated for the pressure filter installations. Pre and post chlorination is recommended with the application of the chlorine dioxide being restricted to the post position.

SURFACE SUPPLY, NO TREATMENT EXCEPTING CHLORINATION

(a) Marginal Chlorination

As only the most excellent surface supply can be used without treatment, marginal chlorination may be satisfactory. The most difficult problem to solve is provision of an adequate retention period. A short retention period may be offset by carrying a higher than normal or free chlorine residual. When a free residual is obtained, the retention period is not as critical, but most surface supplies will create a combined residual and therefore every effort should be made to obtain a good chlorine contact period.

When a good retention period is available in the pumpwell the application point should be the influent end of the well. Continuous operation lends itself to this application point. Where long shut down periods are encountered, or the intake well is very large, the chlorine residual in the pump well could be maintained by a second or standby chlorinator or post chlorination.

When an intake well is not available, the discharge main must act as a contact chamber and the residual should be tailored to provide effective disinfection. When collecting a bacteriological sample from this type system the bottle should contain a dechlorination agent such as sodium thiosulphate. In all cases the minimum chlorine residual is 0.5 ppm.

(b) Free Residual Chlorination

When the ammonia and chlorine demand are low, free residual chlorination is possible. The minimum chlorine residual is 0.2 ppm.

(c) Combined Residual Chlorination

Ammonia can be added at the post position to create a combined residual. Lacking long retention periods it is not advisable to specify a combined residual.

(d) Chlorine Dioxide

Chlorine dioxide can be used for the control of phenolic tastes with the application point being to the pump well.

WELL SUPPLY, NO TREATMENT EXCEPT CHLORINATION

(a) Marginal Chlorination

If disinfection is required because of adverse bacteriological tests, the residual required is at 0.5 ppm after 15 minutes contact between the water and the chlorine. When double pumping is used, the injection point should be to the suction of the high lift pump, or the storage basin. When a sequestering agent is used for iron control, chlorine should be fed to protect the distribution system.

(c) Combined Residual Chlorination

Combined residual chlorination may be required for distribution system control. The ammonia is generally injected before the chlorine feed point. The residual required will depend on the size of the distribution system and the chlorine demand of the water.

WELL SUPPLY, H₂S REMOVAL

Trace amounts of H₂S can be removed with chlorination. When the H₂S concentration is more than a trace, aeration and storage are used to remove the bulk of the H₂S with disinfection and final removal being accomplished with the application of chlorine.

The chlorine injection point is to the high lift pump suction.

WELL SUPPLY, IRON REMOVAL

When open filters or aerators are used, chlorine is applied to the clear water storage or high lift pump suction. Otherwise chlorination may not be required. The minimum residual leaving the plant after 15 minutes contact between the water and the chlorine is maintained at 0.2 ppm. On occasion chlorine may be used to oxidize the iron before filtration.

WATER QUALITY OBJECTIVES --- WHY?

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INTRODUCTION

Historically the availability of good, potable water has been one of the major factors influencing the development of civilization. In probably the first recorded use of treatment facilities, the ancient Chinese and Egyptians used crude methods of chemical coagulation to purify their water.

Modern treatment appears to have had its beginning with the development of the slow sand filter in England in the early 19th Century. It was not until the latter part of the same century, however, that diseases of epidemic proportion were traced to water supplies. Since that time the activities of many thousands of scientists and engineers have been directed toward the development of water purification processes aimed at the elimination of adverse physical and chemical constituents. To meet this demand, more sophisticated treatment methods have had to be developed which have necessitated the provision of more highly qualified operating personnel.

In previous lectures laboratory procedures for the determination of the bacteriological and chemical constituents of water were discussed. In addition, important in-plant and laboratory tests have been described for the daily monitoring of fluctuations in the physical characteristics of the water. The combined results of all of these tests covers the whole spectrum of water quality.

While some of the analytical results have more importance than others, all have certain limiting values which have been established to protect the health of the individual and the well-being of the community. It is the purpose of this lecture to discuss these limiting values, which we shall call "Water Quality Objectives", with a view to their role in attaining the above goals. These objectives were formally adopted by this Commission in 1964 and updated by amendment in 1967. The lecture is divided into three major areas of discussion under the headings of bacteriological, physical and chemical characteristics.

BACTERIOLOGICAL CHARACTERISTICS

There are two methods or techniques in general use in this province for the laboratory examination of samples for bacteriological quality; one is the Most Probable Number (MPN) technique, used by the Department of Health, and the other the Membrane Filter Technique (MF), used by this Commission. These tests have been explained during other lectures in the course and it will suffice to say that, for all practical purposes, the results from both tests are comparable.

Interpretation of Results

There is, however, a slight difference in the interpretation of the significance of actual numbers of coliforms indicated in each 100 millilitre (ml) portion. None of the samples having coliform organisms should have an MPN index greater than ten per 100 ml, while in the M.F. technique none of the coliform counts should be greater than four per 100 ml. If samples of water approach or exceed these limits in consecutive examinations, then an immediate investigation should be initiated to locate the source of the contamination and efforts made to eliminate it. At the same time a series of "special samples" are required to determine the extent of contamination and the progress being made towards its elimination from the water supply. This special sampling is to continue until the bacteriological water quality again proves to be satisfactory. "Special samples" are also required when more than 10 per cent of the samples collected per month and tested by either the MPN or MF methods show the presence of coliform organisms.

A third method of analysis, as discussed in a previous lecture, involves running a series of Presence - Absence tests to confirm the presence or absence of a variety of pollution indicator bacteria. At present this test is only being applied to a limited number of selected water supplies.

Frequency of Sampling

Contamination is often intermittent and may not be revealed by the examination of a single sample. The examination of a single sample can indicate no more than the conditions prevailing at the time of sampling; a satisfactory result cannot guarantee that the observed conditions will prevail in the future. A series of samples over a period of time is thus required.

To ensure reliable results, samples should arrive at the testing laboratory within 24 hours of sampling or be refrigerated if delay is unavoidable. The sample should be collected directly into sterile bottles, not by means of a dipper

or other container. It should be stressed that the reliability of the results is wholly dependent upon the employment of proper sampling techniques and the care with which the samples are collected.

The minimum number of samples required from the source is shown in the following table:

<u>Description of Source</u>	<u>No. of Samples</u>	<u>Minimum Frequency of Sampling</u>
Treated Surface Water	1 raw and 1 treated at plant	once per week
Treated Ground Water	1 raw and 1 treated from each source	twice per month
Untreated Ground Water	1 raw from each source and 1 from each point of entry into the distribution system	once per week

The minimum number of samples to be collected and the frequency of sample collection from a distribution system shall be determined from the following table:

<u>Population Served</u>	<u>Minimum Number of Samples per month</u>	<u>Minimum Frequency of Sampling Intervals</u>
up to 1,000	2	twice per month
1,001 - 100,000	10 + 1 per 1,000 of population per month	once per week
over 100,000	100 + 1 per 10,000 of population per month	once per day

The number of samples determined with the use of the above table shall not include plant effluents whether treated or otherwise.

As a typical example let us consider a municipality of 2,400 persons with two wells providing treated ground water to the system. Bacteriological sampling would consist of the submission twice per month of a raw water sample from each of the sources of supply and a treated water sample from each of

the points of discharge to the distribution system. In addition, a total of twelve bacteriological samples per month would have to be taken from various points in the distribution system. This would entail the submission of three samples per week in order to meet the sampling frequency requirement.

The responsibility for taking the required number of samples, lies with the operating authority whether it be a municipality or an individual who owns a private water supply. The total number of samples to be collected monthly may consist of samples examined by the OWRC, other government laboratories, water works authorities or by commercial laboratories, provided the analytical results are acceptable to the OWRC. "Special samples" shall not be included in the total number of samples required above.

PHYSICAL CHARACTERISTICS

Physical tests do not directly measure the safety of a water supply, however, they do give an indication of its acceptability to the consumer. Thus the objectives that this Commission has adopted to govern the physical characteristics of the water are somewhat less stringent than those required for bacteriological control. The physical qualities which concern water works operators are turbidity, colour, taste and odour. It should be noted that these results are not reported in parts per million (ppm).

Turbidity

Turbidity should average not more than 1 (turbidity) unit. At levels approaching 10 units the water may appear cloudy to the observer. Plants which provide complete treatment should routinely produce water which meets this objective. Ground water supplies will normally meet the objective without the need for treatment.

Colour

Colour should average not more than 5 (apparent colour) units. Colour does not occur too frequently in the natural waters of southern Ontario. However, due to the leaching effect of the water on organic material found in the watersheds of northern Ontario, waters in this region may range beyond 50 units. Removal is possible with alum coagulation, sedimentation and filtration.

Taste and Odour

Taste and odour are very closely related and are caused by the same conditions. The Commission objective for odour is a threshold odour number not greater than 3 and the taste should not be objectionable. Common sources of taste and odour are:

1. dead or decaying organic matter
2. living organisms and oils from algae
3. industrial wastes (phenolic wastes may produce a medicinal taste and odour at concentrations as low as 1 ppb when contacted by chlorine)
4. dissolved gases (hydrogen sulphide, methane etc.)
5. dissolved minerals (chlorides, sulphates and metallic salts such as copper and iron)

The physical characteristics provide only part of the picture of water quality. They are, however, very important when related to surface waters which are of variable quality. Normally ground water will meet all physical requirements, but occasionally the taste and odour of a well or spring supply will be significant. The presence of hydrogen sulphide gas in a well supply in any amount will usually discourage its use.

Turbidity, colour and taste and odour requirements are easily attained during general use by properly designed and operated treatment plants and distribution systems. Failure to meet these requirements is an indication of either inadequate treatment facilities or improper operation of the system.

Consumers will not accept waters which do not meet these basic objectives. They will seek and use an alternate source if it is available. Therefore, these characteristics may be used as a measure of whether or not consumers will accept the product which is being marketed.

Temperature

Temperature is a physical characteristic about which little can be done. The most desirable range is from 40° F to 50° F. Higher temperatures tend to make water less palatable

and reduce its suitability for air conditioning purposes. Temperatures above 80° F are unsuitable and above 90°F are unfit for public use.

CHEMICAL CHARACTERISTICS

Under normal circumstances, analyses for chemical constituents need only be made semi-annually. If, however, the supply is suspected of containing undesirable elements, compounds, or materials, then periodic determinations for the suspected toxicant or material should be carried out at more frequent intervals. On the other hand, where experience, examination and available evidence indicate that particular substances are consistently absent from a water supply as below levels of concern, then, on the approval of this Commission, semi-annual examinations for these substances may be omitted.

Limits for Chemical Constituents

Generally speaking, the chemical constituent concentrations in water may be broken down into two categories; those concentrations which can be tolerated if another, more suitable source is not available and those concentrations which constitute grounds for rejection of a supply. Because of the rapidly changing field of complex chemicals it has become important that the objectives for chemical constituents be reviewed regularly.

The chemical substances shown in the following table should not be present in a water supply in excess of the listed concentrations where, in the judgment of the OWRC, other more suitable supplies are or can be made available.

Substance	Concentration <u>mg/l or ppm</u>
Alkyl benzene sulfonate (ABS)	0.5
Arsenic (As)	0.01
Chloride (Cl)	250.0
Copper (Cu)	1.0
Carbon chloroform extract (CCE)	0.2
Cyanide (CN)	0.01
Fluoride (F)	*
Iron (Fe)	0.3
Manganese (Mn)	0.05
Nitrate (NO ₃)	45.0
Phenols	0.001
Sulphate (SO ₄)	250.00
Total dissolved solids	500.0
Zinc (Zn)	5.0

The presence of substances in excess of the concentrations listed below shall constitute grounds for rejection of the supply.

<u>Substance</u>	<u>Concentration mg/l or ppm</u>
Arsenic (As)	0.05
Barium (BA)	1.0
Cadmium (Cd)	0.01
Chromium (Cr ⁶⁺)	0.05
Cyanide (CN)	0.2
Fluoride (F)	*
Lead (Pb)	0.05
Selenium (Se)	0.01
Silver (Ag)	0.05

* See later section on Fluoride

Problems associated with Chemical Constituents

Alkyl Benzene Sulphonate (ABS)

Contamination of drinking-water supplies with ABS results from its disposal, as household and industrial wastes, into sources of raw water. The concentration of ABS in municipal sewage ranges as high as 10 ppm. Such contamination may appear in both surface and ground water supplies. The objective for ABS in water supplies has been set at 0.5 ppm (mg/l). Tests have confirmed that 1 ppm presents an off-taste to water and at this level foaming often occurs. No apparent toxic effects were evident when tests were conducted with water containing 50 ppm of ABS.

Arsenic (As)

The widespread use of inorganic arsenic in insecticides and its presence in animal foods, tobacco and other sources, make it necessary to set a limit on the concentration of this chemical in drinking water. Our present knowledge concerning the potential health hazards associated with the ingestion of organic arsenic indicates that the concentration of arsenic in drinking water should not exceed 0.01 ppm and concentrations in excess of 0.05 ppm (mg/l) are grounds for rejection of the supply.

Barium (Ba)

This constituent is not particularly common but the Commission has adopted the objective that concentrations in

excess of 1.0 ppm are grounds for rejection of the supply because of the seriousness of the toxic effects of barium on the heart, blood vessels and nerves.

Cadmium (Cd)

Cadmium is recognized to be an element of high toxic potential. Very little attention has been paid to this constituent in the past.

Seepage of cadmium into ground water from electroplating plants has resulted in concentrations up to 3.2 ppm. Other sources of Cadmium contamination in water arise from zinc-galvanized iron in which Cadmium is used.

Tests have shown that concentrations up to 0.01 ppm can be tolerated, but concentrations in excess of this are considered grounds for rejection of a supply.

Carbon Chloroform Extract (CCE)

The Carbon Chloroform Extract (CCE) test is a practical measure of water quality and is a safeguard against the intrusion of excessive amounts of potentially toxic material into water. It is proposed as a technically practical procedure which will provide a measure of protection against the presence of undetected toxic materials.

The test provides an indication of organic material in the treated water, the presence of which shows that pollutants have not been removed in the treatment process. The objective for CCE is 0.2 ppm.

Chloride (Cl)

The objective for chloride has been set at an upper limit of 250 ppm. Above this level a salty taste is apparent. Many municipalities experience higher levels of chloride with varying taste intensities. Abnormal amounts of chloride in a natural water suggests pollution probably of a chemical origin.

Chromium (Cr⁶⁺)

Chromium is another unnatural constituent of water supplies and is not known to be either an essential or beneficial element in the body. Its presence is indicative of industrial pollution probably caused by a plating or tannery operation. The objective in drinking water is such that concentrations in excess of 0.05 ppm as hexavalent chromium (Cr⁶⁺) are grounds for rejection of the supply. Trivalent chromium is not believed to be of concern in drinking water supplies.

Copper (Cu)

Copper is an essential and beneficial element in human metabolism and it is well known that a deficiency in copper results in nutritional problems in infants. Copper imparts some taste to water and is detectable in ranges from 1 to 5 ppm. Small amounts are not generally regarded as toxic but very large doses may cause sickness and, in extreme cases, liver damage. When copper sulphate is used on a surface water supply for algal control, the levels of copper in the water must be closely controlled.

Since copper in small amounts does not constitute a health hazard but imparts an undesirable taste to drinking water, it is reasonable to establish the concentration of 1.0 ppm as the recommended objective.

Cyanide (CN)

Since proper treatment of water will reduce cyanide levels to less than 0.01 ppm this objective has been adopted. For the protection of health, concentrations above 0.2 ppm constitute grounds for rejection of the supply. A very substantial safety factor is provided at the 0.01 ppm concentration which is felt necessary due to the rapidly fatal effect of this chemical.

Fluoride (F)

It has been well established that fluoride in drinking water will prevent dental caries in children, and to a lesser degree in young adults. Where the addition of fluoride to the water supply is practised, a 1.0 ppm concentration is recommended with a permissible operating range of 0.8 ppm to 1.2 ppm, as it is believed that mottling of the teeth occurs at and above this level. When fluoride is naturally present, the concentration should not average more than 1.2 ppm. Presence of fluoride in concentrations more than 2.4 ppm shall constitute grounds for rejection of the supply.

Fluoridated and defluoridated supplies should be sampled with sufficient frequency to determine that the desired fluoride concentration is being maintained.

Iron (Fe)

Iron, at times, is a highly objectionable constituent in water for either domestic or industrial supplies. The domestic consumer complains of the brownish colour iron imparts to plumbing fixtures and laundered goods. In addition, iron appreciably affects the taste of beverages. An upper limit of

0.3 ppm for iron has been adopted as an objective.

Quite apart from the nuisances which are present with a high iron content are the added problems which often develop in the distribution system as a result of the growth of iron bacteria, which thrive in the presence of iron. These present staining problems by converting iron in solution into red, insoluble matter. Watermains may become fouled by the masses of stringy growths associated with these organisms.

It has been pointed out in earlier lectures that iron may be removed by ion exchange, aeration, settling and filtration and iron bacteria may be controlled by chlorination.

Lead (Pb)

Lead taken into the body can be seriously injurious to health, even lethal, if taken in by either brief or prolonged exposure. A concentration of lead in excess of 0.05 ppm has been adopted as an upper limit for drinking water, and concentrations in excess of this amount are grounds for rejection of the supply.

Manganese (Mn)

Manganese presents much the same nuisance conditions as are attributable to iron. It is difficult to remove this chemical and it is recommended that concentrations not exceed 0.05 ppm.

Nitrate (NO₃)

Serious and occasionally fatal poisonings of infants have occurred following ingestion of well waters shown to contain high levels of nitrate. Wastes from chemical fertilizer plants and field fertilization may be sources of such pollution.

Nitrate poisoning appears to be confined to infants during their first few months of life. The nitrates give rise to infantile methemoglobinemia ("blue-baby" condition). It can be cured or terminated by providing nitrate-free water.

This Commission has adopted the objective of 45.0 ppm nitrate as NO₃. In areas where the nitrate content of water is known to be in excess of the listed concentration, the public should be warned of the potential dangers of using the water for infant feeding.

Phenol

Undesirable tastes often result from the chlorination of waters containing extremely low concentrations of phenol. The objective for phenol has been adopted as 1 ppb (0.001 ppm).

Selenium (Se)

Levels of selenium in excess of 0.01 ppm constitute grounds for rejection of a water supply. While trace amounts of this chemical are considered to be essential to man, higher concentrations appear to be extremely toxic in a manner similar to arsenic. Surveys have also shown that selenium may increase the rate of dental caries in permanent teeth.

Silver (Ag)

A water supply shall be rejected if it contains more than 0.05 ppm of silver. This level was established, not because of toxic affects, but due to the unsightly, permanent blue-grey discolouration of the skin, eyes and mucous membranes which may result from its ingestion. Evidence indicates that silver, once absorbed, is held indefinitely in the tissues, particularly the skin.

Sulphate (SO₄)

A laxative effect is commonly noted by newcomers and casual users of waters high in sulphates. A person does however, become accustomed to the use of these waters in a relatively short time. The taste of the water may also be adversely affected. A recommended upper limit of 250 ppm has been adopted.

Total Dissolved Solids

High dissolved solids concentrations are associated with correspondingly high levels of sulphates and/or chlorides. An upper limit of 500 ppm has been adopted as the objective, in order to exercise control over the taste and laxative properties.

Zinc (Zn)

Zinc is an essential and beneficial element in human metabolism and does not appear to have a serious effect on health. The tendencies for zinc salts to impart a milky appearance to water at 30 ppm and a metallic taste at about 40 ppm are the only apparent undesirable characteristics. An objective of 5.0 ppm has been adopted.

Miscellaneous Chemical Considerations

In addition to the purely chemical characteristics which should be considered, there are other characteristics which for want of a better term may be classified as miscellaneous.

Acidity and Alkalinity

Alkalinity and acidity of water refer to the amounts of acids or bases present and are measured in ppm. These are not to be confused with pH which is measured on an arbitrary scale from 0 to 14 and is a measure of chemical activity or intensity. There are no particular limits for either alkalinity or acidity and both are expressed in terms of CaCO_3 .

Acidity is prevalent in many northern waters while alkalinity is a characteristic of waters found in southern Ontario. Acidity is not desirable in a municipal water system, however, it does not normally affect the potability or palatability of the water.

Alkalinity refers to the carbonate, bicarbonate and hydroxide content of a water and is commonly found in the form of a carbonate of soda (Na) and as bicarbonates of calcium (Ca) and magnesium (Mg). Where the alkalinity exceeds the hardness, the presence of basic salts, generally sodium (Na) and potassium (K) are indicated. If the alkalinity is less than the hardness, then salts of Ca and Mg are present in association with sulphates, chlorides or nitrates.

Carbon Dioxide

In surface supplies the normal CO_2 content will range from 0.5 to 2.0 ppm, while in ground water it will range as high as 50 ppm. A proper balance of carbon dioxide in water will ensure that the water is neither corrosive nor scale-forming. This aspect has been discussed in other lectures.

Hardness

The hardness content of Ontario water range from less than 10 ppm to 1,800 ppm. A preferable hardness is in the range of 90 ppm to 100 ppm. Above 500 ppm the water may be considered objectionable for domestic use. Waters with hardness less than 30 ppm are quite soft and probably corrosive.

Hydrogen Sulphide

Even trace amounts of hydrogen sulphide (H_2S) will create a taste and odour characteristic of rotten eggs. It is not harmful from a health standpoint and may be removed by either chlorination or aeration followed by filtration.

pH

Natural waters generally range from 5.5 to 8.6 in pH value. Waters with lower pH tend to cause corrosion and in many cases an upward adjustment to the neutral range (pH 7.0) is necessary.

Phosphate

In a natural, unpolluted water, phosphates have little significance. However, due to the increased use of detergents and commercial fertilizers, phosphates are being discharged into lakes and streams in concentrations which greatly affect biological activities in these bodies of water. Consequently, they exert secondary affects on water supplies, which may necessitate the provision of additional treatment facilities. On the other hand complex phosphates are often introduced into sources of supply for the prevention of corrosion and scaling in water distribution systems.

Radiological Limits

The exposure of humans to radiation is viewed as harmful, and any unnecessary exposures to ionizing radiation should be avoided. Concentrations which exceed, on the average, the values presented in the following table for a period of one year, shall constitute grounds for rejection of the supply.

<u>Radionuclides</u>	<u>Concentration μ μc/l</u>
Radium - 226 (Ra^{226})	3
Strontium - 90 (Sr^{90})	10
Gross beta activity (Sr^{90} and alpha emitters absent*)	1,000

* Absent is taken here to mean a negligibly small fraction of the above specific limits, where the limit for unidentified alpha emitters is taken as the listed limit for Ra^{226} .

Where the total intake of Ra^{226} and Sr^{90} from all sources has been determined, the limits may be adjusted by the OWRC so that the total intake of Ra^{226} and Sr^{90} will not exceed 7.3 micro micro-curies (μ μ c) per day and 73 μ μ c/day, respectively. When mixtures of Ra^{226} , and Sr^{90} , and other radionuclides are present, the above limiting values shall be modified to ensure that the combined intake is not likely to result in radiation exposure in excess of the Radiation Protection Guides recommended by the United States Federal Radiation Council.

FURTHER REFERENCES

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5. Standard Methods for the Examination of Water & Wastewater A.P.H.A., A.W.W.A. and W.P.C.F. (1th ed., 1960).
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ELECTRICITY AND ELECTRICAL MOTOR MAINTENANCE II

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REVIEW

At the Intermediate Course (all of you must have passed to be here) you heard some small part about electricity. Firstly, we shall briefly review the quantities of electricity as compared to water:-

Volts can be compared to pressure
Amps can be compared to flow
Watts can be compared to horsepower

Watts = Volts x Amps
Horsepower = Pressure x Flow

Two kinds of flow in water would be steady flow and intermittent flow. In electricity, it is listed as direct and alternating current flow. In both electricity and water, both types of flow can be used to do work.

In electricity, a phase means a time, therefore, three phase power means power that reaches a peak at three separate times. This timing is built in at the source of power.

POWER FACTOR

Now we have another term in electricity called Power Factor and it, too, has an equivalent in waves in water. A wave breaking directly on a pier in the water exerts all its power against the pier. With electricity this would be high power factor, because the electricity is working directly at the load. If the pier were at an angle to waves, the wave would be pushed sideways and only part of its force would be used against the pier. In electricity, this would be a medium power factor. If we moved the pier so that the waves almost ran along it, there would be very little force against the pier, and, in electricity, the power factor would be poor.

The type of load determines the power factor, just as the angle of the pier determines the wave action against it. Electrical loads that produce heat, such as incandescent lamps and heaters, are good power factor loads.

Motors can be either medium or high power factor depending on the load and the design of the motor. A motor with too small a load gives a poor power factor, and its power factor improves as the load reaches the load for which the motor was designed. Special large motors are made that produce an excellent power factor.

Good power factor means that the user gets all the power he pays for. Poor power factor means usually poor output for the user because, like the wave working on the pier, the work is there in the form of the wave, but it passes by without doing any work.

In electricity, power factor can be improved by adding capacitors which make the electricity do more work at the same reading on the meter. In large loads and steady loads, such as water pumps, the use of capacitors saves money for the user. Usual practice is to design capacitors to repay their cost in about four years.

LOAD FACTOR

Another term used in electricity is load factor, and this means that you use the power as evenly during the whole twenty-four hours of a day as possible to get a good load factor. High load factor and high power factor result in the most economical use of electricity, and an effort should be made to reach as high a level as possible.

MOTOR CIRCUITRY

Now to talk about circuits for a short time. Electricity has to travel in a complete circle, or it doesn't travel at all. When you open a switch you stop the circle just like a valve stops the circle in water, and, as in water, when the circle is broken, no electricity flows. If you close a valve in water, the water ahead of the valve still wants to leak out as long as the pressure is there, and so it is with electricity.

If you open a switch in electricity, the power is still there ahead of the switch -- you can see the water leak, but just remember the power is ahead of the switch and make sure you are not the electricity leak. Do not let the power flow through you.

In medium and small-sized pumping plants there are three common electricity supply voltages, 4,000 volts feeding into extra large motors, and at the same time it feeds transformers to operate smaller motors and lights and equipment. The transformer reduces the voltage (pressure) and increases the flow to produce safer working voltages. Medium sized pumping plants expect the people who supply the electricity to furnish power to fit the load, so the voltage is supplied at 600 volts for the motors and another supply for the lights at 120 volts.

The 600 volt power is three phase and the 120 volt is single phase. The reason three phase power is used is fairly simple. More power can be delivered for the same size wire and a motor on three phase can be started under full load without extra equipment. Thus, a pump motor on three phase can be connected to stop and start with very little trouble a very great number of times. If a single phase motor were connected to do the pumping, it would require auxiliary starting equipment because its starting power is not as great as its running power. The cables feeding the single phase motor would be much larger than the three phase cables and the motor in large sizes is very much more expensive than the same size in three phase.

MOTOR MAINTENANCE

You men work at plants that are now in operation, so that unless there is a general trouble, you should consider that electrical failures will be the result of mechanical troubles, such as bad bearings or shift of alignment. These will overheat the motor. The second main cause of trouble is moisture - a broken water line sprays water, or a pump seal leaks and sprays the motor. If a motor is really soaked, do not operate it until it can be dried and tested. Drying a motor takes less time and is much cheaper than burning it out because it is wet.

SAFETY DURING MOTOR REPAIR

Finally, use your head to make sure at all times you are safe. If you want to work on a motor, lock the switch open. On some switches you can put a padlock on the switch so it cannot be closed - that's cheap insurance for you to buy your own padlock and lock the switch open when you are working on the pump or the motor. Please be careful!

BUILDING AND GROUND MAINTENANCE AT PLANTS

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A prime requisite for any plant treating or pumping water, is that it be clean. There is an immense psychological value keeping plant in a high state of cleanliness. Visitors do not expect a high standard of cleanliness in a plant and it is not unreasonable to assume that many visitors come in a critical state of mind. If your plant is clean, these visitors are pleasantly surprised. This is lesson I in public relations. Remember the quality of your product is measured by the appearance of your plant. Dirty plant, dirty product. Good plant, good product. We will discuss ways and means of creating this correct impression with the public. It will be discussed in two distinct parts:- Building Maintenance and Grounds Maintenance.

A - Building Maintenance

A large part of this is housekeeping. Cleaning should be done on a routine basis. Particular emphasis should be placed on office space, lunch room, washroom and locker room. Close attention should be paid to laboratory facilities and the control area. As well as being kept clean, these areas should be kept tidy. As a matter of fact, if you haven't time to keep the place clean, keep it tidy. Next in the order of importance and areas such as filtered galleries, pump rooms, chlorine rooms. Wherever possible floors in these areas should be kept waxed and polished. A non-skid wax should be used at all times. A hospitalized visitor isn't likely to be too complimentary. Keeping floor areas clean in the winter time is a problem, especially if the operator has to run in and out answering the telephone and fetching tools, etc., but must be done. The floors can be protected by coconut matting paper or rubber runners in the main tracking areas. In the modern plant, an industrial type floor polisher is almost a necessity and if you do not have one you should make every effort to persuade your employer of the need for one. Next in the order of importance and forgotten by most but the visitors, would be the window ledges, desk tops, and shelving. These should be kept clean at all times. Some visitors like to look in cupboards. Keep in mind that cupboards are for storage and not for depositing odds and ends in a haphazard fashion. There is a bonus to the operator, in this, in that he knows where things are. Next on the list would be the windows. Windows are for two basic purposes:- one, to permit light to enter the building and to enable one to look out. When windows become dirty, efficiency in both these

items is lost. It should be noted that when windows consist of two sides, inside and outside, both get dirty.

The operator can be aided in his cleanliness by routine painting and waxing. Scaling pipe never looks clean and an unwaxed floor is difficult to maintain. This waxing of floors can be carried one step further. In recent years, waxes have been developed for concrete floors and these waxes can be applied successfully in pumping station dry wells and basements of control buildings. The basic reasoning is sound. Any concrete floor will continually dust from the day it is poured. Dusting becomes worse as time progresses. The wax seals the concrete from the eroding action of the atmosphere. A good concrete sealer will have the same effect but it will not shine up under polishing as is in the case of application of liquid wax. In one large pumping station where the operator brushed the floor daily, he found that after applying proper wax and polishing that his maintenance was reduced to hosing down once per week. We now move on to the laboratory. Larger plants are now equipped with sizable laboratories, and even the smaller ones have some designated area where testing is done. Glassware on counters and racks should be kept clean when not in use. Solvents are available for cleaning glassware.

Equipment painting is a subject by itself and this lecture will confirm itself to the basic appearance rather than the protection aspect. The ground rules apply:

1. Do not paint over rust.
2. Do not paint over partially scaled paint and if paint continually peels, find out why.
3. Paint work should almost always be cleaned with a stiff wire brush or sandblasted, wiped, primed and finished off with an application of finish coat.

There is a fairly general opinion that copper and brass fittings and piping should be burnished and not painted. If they are burnished, a light coating of clear varnish will maintain the appearance indefinitely.

Painting of concrete floor can be a risky business. What will give satisfaction in one plant gives heartache in another. However, concrete which has been well cured and has received a finished surface, receives paint readily and general appearance is satisfactory.

The following are a few tips for aiding cleanliness and easing operation. A simple wall rack for the storage of current magazines does much to improve the appearance. If your plant has a lunch room, dirty cups and cutlery should not be left around but should be washed and returned to

storage after use. Lockers should be provided for street clothes and personal belongings and protective clothing should be stored on a rack close to an outside door.

If you have a workshop area this should also be kept clean. Mechanics, although they take great care of their tools, tend to be untidy. They should be encouraged to clean up after extensive use of their workshop, put away dirty rags, sweep the floor and store bulk equipment neatly.

A word of caution here, if there is a janitor at your plant, don't try to make him into a personal servant. He has lots of work to do.

There are numerous side benefits from this public relations approach to cleaning. There are better working conditions for yourself, things are easy to find when you need them, and safety practice is made easier.

Still confining our discussion to buildings, we will examine what can be done to aid the appearance of the outside of the building with a minimum of additional effort.

When we discussed windows, we managed to get both sides clean so we will leave them now. Brickwork, when it becomes splattered with mud or simple rain soils should be hosed and occasionally brushed. It may be necessary, on occasion, to wash the brickwork down with muriatic acid, especially if efflorescence is the problem. Where vertical wood siding is used as an architectural feature, it is much easier kept oiled than varnished. If your not already keeping your tanks hosed and brushed it is too late to mention them. However, here are a few thoughts on launders. Launders can be painted with either a metallic or base paint. Good use can be made of ceramic tile, especially in white or pale colours. This is not as ridiculous as it sounds because it makes your final or interim product appear much cleaner. Whatever your choice of paint or tile they are much easier kept clean and in many cases, a rubber squeegee can be used in preference to a wire brush. Hand rails around tanks should be kept clean and painted and lucky is the operator to inherit aluminum hand rails. In the larger plants, a steam jenny is a good investment. All this endeavour would appear to be time consuming, and it is if you have to start from scratch, but the plant Superintendent or Chief Operator who has had his plant spotless for an official opening, will confirm the old adage that it is easier to keep a clean plant clean than to keep a partly clean plant looking partly clean.

Grounds Maintenancea) Grasscutting

Area is the criteria. If you have a relatively small area, it can receive complete attention and be cut regularly. However, if you have a large area, you can go broke on cutting grass alone. Let us consider a small area to consist of one acre or less. Here there is no problem since an area of this size can be kept close trimmed and weeded at all times. A word of caution here. Don't cut your grass too short in summer time since all you will do is burn it out. Where you have larger areas, in many cases, up to ten or more acres, the operator does not have the time nor staff to keep the entire area in bowling lawn condition. Where this is the case, attention should be concentrated on the area adjacent to buildings, tanks and access roads. This area should extend some 25 to 50 feet beyond buildings and tanks, and in the case of access roads, nothing up to 25 feet on either side of the access roads clipping the ditches, should be kept trimmed and weed free at all times. Grass beyond these areas should be kept reasonably short and cut down when it grows to or reaches the height of approximately 6 inches. These fringe areas should not be watered and in this way the grass will remain green and it might be possible to cut no more frequently than once every two weeks. These fringe areas in most plants would extend to the fence line or even beyond. At many plants, property beyond the fence is owned and if this is a field, consideration should be given to renting out or lending to a farmer.

Lawn-cutting Equipment

Much has been said on this and much will be said and it gets down to the matter of individual choice, however, here are some basic guide lines. For steep banks around tanks or steep contouring, a small two cycle mower is a must. Its also handy for ducking in and out of flower beds. On the larger, flatter areas, a large rotary four cycle mower is more desirable, something of the size of a $7\frac{1}{2}$ - 10 HP Gravely, either walking or riding. The thing to look for here is a wide cut, 54" and more is now possible with double sweep rotary blades. This will handle most of your grasscutting and will cut the working time substantially. If your plant is located in what looks more like an estate than a piece of property, large farm type tractor type mowers should be considered. Although they are expensive, they will eventually pay for themselves and save time and labour.

b) Landscaping

The immediate building area extending 25 to 50 feet beyond the buildings and tanks should be landscaped. As previously mentioned, grass should be close cut and weeded. But again a word of caution, do not cut grass shorter than an inch and a half. In this way, the grass develops a strong root structure and will choke out weeds by natural process. The use of gravel or crushed stone adjacent to the tanks, is serviceable but mainly to the growth of weeds in the gravelled area. This can be overcome by laying slight gauge plastic under the stone. Lawn areas are the most attractive and eye-catching and these should be kept clear of weeds as previously mentioned. There are numerous selective weed killers on the market and the operator has a wide choice of what he wants to kill. In this he is free to kill everything but the grass.

If water is available, the areas immediate to the building and tanks should be kept watered. However, during periods of summer water shortages, it does not look good for you the plant to be using water when everyone else is on restrictions. This will nullify your public relations.

Access roads, pathways and driveways are frequently neglected, often through no fault of the operating staff. It is impractical to have a gravel driveway on which trucks are making several daily trips. After several years of this operation, the access road assumes the appearance of a rutted cow path and very little can be done except regrade it. Under these circumstances, lightly travelled roads should be oiled and heavily travelled roads should be paved.

Flower beds enhance the plant appearance but are time consuming and should be kept to a minimum and restricted to locations where they immediately catch the eye. In planning flower beds, do not restrict the flowers to one space. Arrange them so that there is a group of flowers in bloom throughout the summer season. Perennials frequently require less attention and less financial outlay than the annuals. Trees and shrubs should receive close consideration especially beyond the immediate landscaped areas. Trees have three basic functions: windbreaking, screening and sun shade. As you all know, there are two basic trees in this area:-

a) coniferous

b) broad-leafed

(a) Coniferous trees would be -

1 - Eastern White Pine - which is a tall, stately tree and can reach a height of 175 feet with a trunk diameter of 5 feet, but this is unusual and it is usually 100 feet or less in height and no more than 3 feet in diameter. This tree will grow on wide range of sites from dry sandy to bogs, but it does best on a moist sandy soil.

2 - Red Pine - Its average height is 100 feet with a trunk diameter less than 3 feet. It grows best on a loamy sand or gravel but can thrive just about anywhere.

3 - Tamarack or Juniper - This is a medium size tree 60 to 70 feet high with a trunk diameter up to 2 feet. This tree grows best in damp, swampy areas.

4 - White Spruce - This can grow to a height of 120 feet with a trunk diameter of 4 feet (usual height is 80 feet). This tree grows best in well drained moist soil close to streams.

5 - Hemlock - This is a medium size tree growing to some 60 to 70 feet in height and up to 2 feet trunk diameter. This will grow just about anywhere.

6 - Balsam Fir - This also is a medium size tree and it will grow just about anywhere.

(b) Broad-Leaf Trees

1 - Aspen or Poplar - The average height of this tree is about 40 feet with a trunk diameter of about 8 to 10 inches. This tree grows best on a well drained loam and is found almost anywhere.

There are other broad-leaved trees, some of which are:- Walnut, Hickory, Iron Wood, Birch, Beech, Oak, Elm and Maple.

I have mentioned only a few of the possibilities. There is an infinite variety of trees, but the ones that I have mentioned are readily adaptable to Southern Ontario climates and the Evergreens, of course, will grow to the far North of Ontario. Operators with plants on the North shore of Lake Superior should take the advice of the local Forest Ranger. Broad-leaved trees should not be planted adjacent to tanks or they create a leaf problem in fall with subsequent plugging of pipes and pumps.

Those operators with small plants will not be able to go into the tree growing aspect and should consider instead, the growth of shrubs. Here again, climatic conditions prevail, but there are numerous Evergreen shrubs that can be grown easily in Southern Ontario.

LAWNS

(a) I MATERIALS AND TOOLS

1 - Humus and fertilizer - Any texture of soil will suffice in a pH of between 6 and 7 if necessary. Starting with 50 lbs. of fertilizer per 1000 sq. ft. - first year - it should be fed at the rate of 20 lbs. per sq. ft. per year, preferably in March, April and September.

2 - Grass seed of good grade in a mixture suitable to soil and light conditions - Quantity -- about 4 lbs. per 1000 sq. ft. Tools and machinery -- a Roto-tiller; spade, if the work is to be done by hand; iron rake; baskets, wheel barrow, roller or tamper; and a hose.

II PROCEDURE

(a) Building a New Lawn: -

1 - Choose the most opportune time to sow seed. Early fall, late August or September is best, with early spring as a second choice. Rye grass or Timothy can be sown temporarily - to be ploughed or dug under later, or sodding may be resorted to - although it costs twice as much as seeding.

2 - Prepare the seed beds some weeks in advance so that it can settle before sowing.

(a) - Test soil to find out if lime is needed and how much. (Check the pH)

(b) - Plough or Roto-till the existing soil.

Remove stones and debris and rough grade before preparing the seed bed.

(c) - Replace any top soil which may be piled to one side of the lot during rough grading for drainage, etc.

(d) - Spread (all at one time, if desired) any necessary lime, commercial fertilizer humus and mix them with the top 6 inches of soil by Roto-tilling or digging it at least a week in advance of seed sowing.

3 - Other good grade of lawn seeding have a mixture which suits soil and light conditions. If the grounds are in part shady areas, then a shady mixture may be sown throughout. If both sunny and shady mixtures are being used, grade the two together where they meet to avoid the deciding difference in grass texture.

4 - Add more fertilizer to the surface inch or two, rake it in and smooth down the surface just prior to seed sowing.

5 - Sow the seed in two directions, first splitting the quantity equally.

6 - Rake the seed into the soil very lightly after sowing and firm the bed with a light roller, if available, or a tamper.

7 - Water with a spray mist if drought sets in before the seed sprouts well. Supply ample water throughout the first season.

8 - Begin to mow when the grass reaches a height of 3 or 4 inches. Set the mower to mow the new grass at $1\frac{1}{2}$ inches height.

9 - Destroy crab grass and other weeds when they appear. Do this by hand the first season as chemicals would be too strong for new grass.

10 - Follow through every year by feeding and top dressing in the spring or fall and by keeping the grass as weed free as possible.

(b) Renovating an Existing Lawn: -

If a lawn has a stand of $1/3$ or more of good grass, it is worth renovating. The following method can be used to improve a new lawn if poor thin soil, or an old lawn which has become crab grass infested.

Early fall is the preferred time to renovate, otherwise early spring.

1. - Mow the grass short first then rake off the clippings along with the fallen leaves, dead crab grass and other weeds.

2. - Feed the whole lawn with a special lawn fertilizer or general commercial fertilizer.

3. - Scratch all the bare patches of soil with an iron rake.

4. - Seed them with a good mixture by hand and rake in very lightly.

5. - Using a light roller or tamper, compact the soil.

6. - Keep the surface soil moist by watering, both before and after the grass shows above ground.

Now in conclusion: We have not covered the entire field. The purpose of the lecture was not to tell you what to do and when to do it, but rather to make you see the overall problem and plan your attack accordingly.

EFFECTIVE PUBLIC RELATIONS

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Public relations by a water utility is communication with the general public and employees of the water works. It is an attempt to keep these people well informed of the accomplishments, developments, problems, needs and objectives of the public utility. In essence, it advertises the fact that it provides an excellent service so that it receives the appreciation it deserves and hence the good public image of the utility is enhanced. It will then have to maintain a constant vigilance of its liaison with the public to protect the good image it has created.

There are many reasons for maintaining a good public relations programme but, the most general is that it is "good common sense". The consumer wants to know how his dollar is being spent and in some cases the opportunity to vote on the acceptance or rejection of a major project. Of course there is a limit governed by the size of the municipality and necessity of the project whether their opinion is sought or not.

The manner in which such a programme is executed is dependent on various influencing factors in the municipality. For example, smaller municipalities are closer to the public and are able to deal with problems on a more personal basis than a large municipality. It is left to the discretion of the management of each water works system to decide on the most effective approach.

An effective public relations programme is often a direct reflection of management's ability and interest to get along with the consumer and general public. This fact may be noted by an industrial representative who is travelling through the town. He may be sufficiently impressed to consider that town for the location of a new plant. He realizes that good management is a reflection of good business. Hence, it can stimulate the economic growth throughout the entire community and in this way help to build and secure profit for all its inhabitants.

The discussion has so far been limited to the effects of external communications. It is just as important to maintain good internal communications or management-employee liaison.

It is "good sense" to develop in the employee a loyalty to the organization and an understanding of the goals it hopes to achieve. In this way the productivity of the employee is stimulated and he will work to his maximum level of efficiency to provide the best service he can. The realization of this fact is good management technique.

There are countless methods being used by municipalities in maintaining an effective public relations programme. Some of the rules to follow in accomplishing such a programme are now presented. We will deal with both aspects of communications, internal and external, since the effective execution of the latter is somewhat dependent on the former.

INTERNAL COMMUNICATIONS

In-service Training

A new employee whether it be a secretary, repairman, meter reader or plant operator should be properly instructed of the goals and purposes of the water works. The history, role, general operation, policies and future developments of the system should also be accurately outlined verbally and in written form by the supervisor or personnel officer of the employee. In many cases these employees will be your first-line of contact with the public.

Each man and woman in the utility family should know the component parts of the utility and their relationship to one another. They should be made to realize that they are an integral part of the organization and help make the utility operate. If fully informed of the operation of the water works, the employee can accomplish a great deal in promoting public understanding and appreciation of the efforts of the utility.

In addition, all the other employees should also be kept up-to-date of recent developments, problems and future proposals.

One method of transmitting this information is by publishing a newsletter on a regular basis. Bulletin boards should be available in employee traffic areas where news bulletins which affect the status of the employees can be placed.

Retired employees should also be sent copies of these employee publications. They too may be interested in the progress of the water works and can be utilized as valuable assets in the

public relations field.

Routine Management-Employee Meetings

Regular discussion sessions should be held between the management and the other employees of the water works. This enables all the workers to openly voice their complaints, express their views, suggest ideas and generally keep abreast of any proposed modification of the water works or organizational structure.

The provision of a suggestion box is another method of allowing employees to list their complaints on record.

Public Relations Seminars

The purpose of a public relations seminar is to train the personnel in this aspect of their occupation.

Secretaries and switchboard operators should be taught proper telephone courtesy when talking to the public. They should be knowledgeable of all general particulars of any detours caused by repairs or installations of water pipe lines. She should know how long the public will be inconvenienced, the capacity of the new main, the name of the contractor who should be contacted if more pertinent data is required, etc.

Meter readers and repairmen should especially be well acquainted with the operation and progress of the utility. They come into regular direct contact with the consumers and hence may be asked to answer many pertinent questions. They should also realize the importance of proper etiquette, cleanliness, manners and neatness when dealing with the householder. Some plants provide uniforms for their employees and this practice merits careful consideration by the utility managers. This is to protect the customer from those who would represent themselves as water works employees to gain entrance to the property.

Selection of Duties for Operators

All the operators at a plant should be informed of the mode of operation of all various units at a water works and their function in the treatment process. In case of an unexpected visit by a civic official, government representative or any other official delegate of some agency, anyone in the plant would be able to direct the plant tour.

People naturally expect the water works and its surroundings to be continuously kept in a neat and clean condition. They may not know the inner workings of a utility but they do take pride in an attractive water works plant. It is up to the operators to keep it so.

In selecting the various duties for the operators, the supervisor should be aware of any particular interests of the employees. For example, if one has talents in gardening, painting, photography or any other related interests or hobbies that would be useful at the plant, this should act as a guide in the selection of his duties. It will give the individual a greater pride and inspiration in his work if it is tailored to suit his interests.

EXTERNAL COMMUNICATION

We have discussed several methods of maintaining an internal communication within the utility. Now with a well instructed staff, this atmosphere or knowledge will tend to permeate to the general public.

Keeping the Various Public News Media Informed

Most news media welcome the help of legitimate publicity, which, for the most part, is beneficial to both parties. The utility should take advantage of any good publicity they can obtain. The common news media that can be accredited for this publicity are the local newspaper and radio and television stations. They are interested in personnel changes, transfers, awards, plant tours, open houses and other events of interest. You can assist men associated with these media by submitting items which you are reasonably sure are of public interest.

You should maintain a good contact with members of these news media. One man from the plant should be selected to be responsible for publicity. The utility should help him in developing an effective technique in writing or giving interviews. He may also assist in the production of a regular newsletter or other work of this nature.

Customer Complaints

Customers must be able to reach the utility to voice their attitudes and complaints. The response in all these circumstances must be friendly and patient. The complaints should

be effectively handled as soon as possible and, if need be, followed up at a later date.

Telephone manners when dealing with a complaint cannot be overemphasized. The dissatisfied customer will tend to be less harsh when the person he is talking to radiates a spirit of warmth, enthusiasm and understanding over the telephone. It isn't intended to imply that one should be artificial in such a case. On the contrary, he should appreciate the problem the complainant is experiencing, be able to discuss the situation intelligently and try to offer an immediate (even though it may be a temporary) solution to the problem. Ideally, in some cases, it may appear that you require a machine to possess the above qualities when dealing with an irate customer. Not really - but it does require the selection of a patient person who has a genuine feeling for his job and a naturally gifted knowledge of human relations.

A factual record of the conversation should be made and passed on for processing.

Proposing an Increase in Water Rates

Eventually a water rate increase will be imposed on the users. A good service record and a knowledgeable public gives convincing support to the technical case for a rate rise. In the event of a proposed rate increase the following steps are suggested:

1. A postcard notice is sent to each customer and a letter to each employee outlining the reason for the increase.
2. The necessity for the increase is publicized over the various news media.
3. A more detailed explanatory letter is sent to each large consumer.
4. Local industries and hospitals are notified as well in advance as possible to allow them to consider the budgetary impact of the increase.

The utility may also include a chart showing the progress of the water works, the financial comparison of operation, between various years, a distribution of the revenue dollar and the increased water use and pumpage over the years with their letters to the consumers. A present or proposed map of the area and population to be served should also be included to enable the

people to realize the tremendous water demands of the growing municipality.

If there is a choice to either (1) impose no present increase in water rates and deliver a water of past quality or (2) to impose a pre-calculated rate increase and serve a water of improved quality, the decision of the consumer can be determined by conducting a postcard survey.

Signing

A courteous maneuver, if a road is to be torn up for the installation or repair of water mains, is to conspicuously locate signs apologizing for the detour. Also inform the traveller as to the length of time the project will take, the reason for the detour, etc. If you are expanding the size of the main, give general particulars of the new size of pipe, its increased capacity, and who to contact if more information is required. This is why, as mentioned earlier, the telephone switchboard girl should be familiar with the project. This approach will be one of the most effective ways of informing the public of the progress of the community.

The water utility should consider designing a symbol to represent the water works and using it wherever applicable. It should be borne in mind, however, that the OWRC symbol may not be used for any purpose by anyone other than Commission personnel.

Plant Tours

Plant tours or open houses are most essential in creating a desirable public utility image. Open houses are generally held to celebrate a plant birthday, community anniversary or some such event and the entire community is invited to visit the plant. Plant tours are arranged to appeal to a select group such as teachers, students, civic groups, members of organized clubs, etc. Programmes for these tours are designed to fit the interest of the specific group.

Tours should be encouraged by the plant operators and management. In addition to the obvious public relations benefits of conducting a plant tour, it stimulates and encourages the operators to keep a neat house and well maintained equipment.

When planning a plant tour set a definite theme and incorporate your objectives but don't confuse the visitors. In

general, give broad impressions and keep the programme simple. Remember the purpose of the tour is to teach and not to dazzle the visitors with your knowledge and technical terminology of water treatment. Some points to keep in mind when planning a tour or open house are given below.

1. General Housekeeping - Keep the yard, entrance and plant clean and neat. The erection of a welcome sign is a friendly gesture.

2. General Description - Prepare the visitors for the tour by describing the general treatment process with reference to a wall-size schematic diagram which should be available at the plant. The function of each unit can be commented on while using this visual aid.

3. Groups - Split up the group into smaller parties, preferably of six or seven people, or depending on the number of guides available. The guides will probably consist of the plant operators. Therefore they should be thoroughly familiar with the operation of the plant and treatment units. If necessary use a film to describe the process.

Routing

Signs with arrows showing the simple direct route to be followed by each group should be established. It may be helpful if a short explanation of the function of each unit is described on a placard and displayed at the respective units.

Transportation

Arrange for ample parking space to be available to accommodate the cars and/or buses required to transport the guests.

Souvenirs and Refreshments

Inexpensive souvenirs (key chains, note books, plastic drinking cups with the water works symbol, etc.) may be given to the groups as a tangible reminder of their tour and help to create a good feeling towards the utility system. Some refreshments may be served if practical. They don't have to be elaborate. At this time, a period could be planned to discuss their tour to somewhat more detail now that they are more knowledgeable about the treatment process. The guests should be requested to sign a visitor's book.

Promotion and Publicity

News stories accompanied with photos of the group and follow-up letters to the group representatives should be carried out. It may be an idea to take a photograph of the group and send them a copy, when developed, and also pin a copy on the visitors' bulletin board.

General Assessment of Tour

The tour should be evaluated carefully and objectively by the organizers so that the subsequent tour can be improved.

CONCLUSION

The object of this presentation is to make one aware of the need of an effective public relations programme for the water utility. Several methods of augmenting the present programme in effect in a municipality are presented. It is by no means implied that the programme is restricted to the methods discussed. There are virtually countless ways of promoting good public relations and the most effective methods may be different in each municipality.

You may find that, in the initial stages, the effort will be costly but, ultimately, is less expensive than to permit people to form false impressions and distort the facts through ignorance. One of the best ways of maintaining a good public image is to keep the people well informed as to the manner in which their money is being spent. Public relations is adequate when the people are confident that the utility's programme is sound in engineering and economically feasible.

You should make a personal evaluation of the effectiveness of the public relations programme established by the water works in your community.

BACTERIOLOGICAL EXAMINATION OF MUNICIPAL DRINKING WATER
SAMPLES BY THE PRESENCE-ABSENCE TEST FOR POLLUTION

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Division of Laboratories

INTRODUCTION

The Presence-Absence (P-A) test for pollution indicator organisms is primarily a simple modification of the Most Probable Number (MPN) method. The MPN coliform procedure was in common use for many years before the introduction of the Membrane Filter (MF) technique. The MF technique, although providing a more rapid and precise method of determining pollution in surface waters, has been shown to lack sensitivity in detecting coliform bacteria in treated or chlorinated waters.

The Presence-Absence (P-A) test, as the name implies, determines only the presence or absence of pollution organisms, but does not measure their numbers. In municipal drinking water supplies, the presence of any coliforms in 100 ml. of sample indicates that the water is unsuitable for drinking purposes. A positive result for pollution in a sample may be due to inadequate treatment of the water supply, contamination of the distribution system or even faulty technique in taking a sample for laboratory analysis.

COMPARATIVE EVALUATION OF P-A TEST WITH MF AND MPN TESTS

A number of advantages may be listed for the P-A test over the conventional MF and MPN procedures.

1. The P-A test has been shown by extensive laboratory analyses to be more sensitive in detecting coliforms and other pollution indicator organisms than the MF technique.
2. Pollution indicator bacteria, other than coliforms include the fecal streptococci, fecal pseudomonas and fecal clostridium organisms. These bacteria may be detected more easily by the P-A test, than by either the MF or MPN procedures.

3. The potential advantages in having a number of bacterial groups to indicate pollution has particular merit when coliforms are absent or present in small numbers in a drinking water supply.

Two changes in the usual manner of reporting analyses, which may at first be considered as disadvantages will be:

1. The reporting of the presence or absence of coliforms (and/or other pollution indicator organisms) rather than a numerical value per 100 ml. of water sample.
2. An extension of the test period from one to five days before reporting water samples as negative for evidence of pollution.

In the latter instance, comparative MF and P-A analyses have shown that when coliforms are present in small numbers, or have been devitalized by water treatment processes or unfavourable conditions in the sample bottles during shipment to the Laboratory, they may remain undetected by the usual 22-24 hour MF test. These coliform bacteria frequently recover and produce a positive reaction by the P-A test during the extended incubation period.

PRESENCE OF POLLUTION

If the P-A test gives a good positive result in 24 to 48 hours, then either large numbers of coliforms or fresh pollution is present in the sample. The municipality will be notified of the unsatisfactory sample(s) and will be requested to take additional samples. Repeat samples from locations, which have given a positive test for pollution in the preceding sample(s) should be marked as "P-A positive". These repeat samples will receive both a P-A test and a MF test.

Municipalities should be aware that their raw water samples will continue to be analyzed by the Membrane Filter (MF) procedure to enumerate any coliform bacteria that may be present. In addition, depending on the relative water quality by previous analyses, at least 10% to 20% of their drinking water samples will receive

MF analyses to protect against emergency situations and to monitor the general bacterial quality of the water when coliforms or other pollution indicator organisms are absent.

REPORTING RESULTS OF P-A TESTS

The pollution indicator bacteria which may be reported by the P-A method of analysis include:

1. coliforms (Escherichia, Enterobacter, Citrobacter and others)
 2. fecal coliforms (Escherichia coli and others)
 3. fecal streptococci (Streptococcus faecalis and others)
 4. fecal pseudomonas (Pseudomonas aeruginosa)
 5. fecal (anaerobes) clostridium (Clostridium welchii)
- the latter four groups require special confirmatory tests
 - under suitable conditions, coliforms, fecal streptococci (enterococci) and fecal clostridial bacteria produce distinctive reactions in the P-A bottles.

INTERPRETATION OF RESULTS OF P-A TESTS

- (a) The presence of fecal coliforms, fecal streptococci or fecal pseudomonas signifies water of unsuitable and dangerous quality if used for drinking or washing purposes without remedial treatment.
- (b) A sample producing a confirmed positive P-A result for coliforms within one to three days indicates water of unsuitable quality for drinking purposes.
- (c) When a confirmed positive test for coliforms has been obtained after four to five days, this result represents water of doubtful quality and another sample should be taken to make certain that the water quality has not deteriorated further.
- (d) Fecal clostridial bacteria are anaerobic spore-formers, generally more resistant to chlorination than other pollution indicator organisms. Their presence indicates the water has been previously exposed to fecal pollution, but has probably been rendered safe from the disease-producing bacteria. Under certain conditions, Clostridium welchii (perfringens) has been implicated in outbreaks of food-poisoning.

SUMMARY

The P-A test provides a simple, sensitive test for a wider selection of pollution indicator organisms than was previously possible by other methods. Close co-operation between the Laboratory Analyst and the Sanitary Engineer will permit notification of polluted samples to the municipality soon after they have been observed. A more comprehensive pattern of the municipality's water quality will be achieved by monitoring its water supply for a range or spectrum of pollution bacteria, rather than by only the coliform group.

SAFETY IN THE HANDLING OF CHLORINE

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There is an old saying - "familiarity breeds contempt". Certainly familiarity with Chlorine and a knowledge of its characteristics is highly desirable but I hope that this would not breed contempt. I would rather like to say "familiarity breeds respect".

I am afraid that this talk like many others I've heard on safety is going to deal with potential hazards rather than foolproof methods of handling and this is because there is no such thing as a foolproof system.

Even with the best of precautions, accidents do happen and it is, therefore, especially important that all people working with or near Chlorine should be familiar with its properties and, above all, not to panic if exposed to an emergency situation. A brief review of the history, characteristics and properties of Chlorine will help us understand the product we are dealing with, thereby enabling us to handle and use it more competently.

Chlorine is one of the most important chemicals of our modern industrial world. It was identified as an element in 1810 but commercial quantities were not produced and shipped until 1888 when it was found that the dry Chlorine gas did not attack iron and steel. The Canadian demand for Liquid Chlorine has increased from approximately 10 tons per day in 1922 to over 1400 tons per day at the present time, with water and sewage treatment representing a substantial portion of this tonnage.

Chlorine is prepared from a salt brine by an electrolytic process in which the brine decomposes to form Chlorine, Caustic Soda and Hydrogen. Chlorine is liberated at the anode, a positive electrode of the electrolytic cell, and drawn off through stoneware pipes to the drying and liquifying plant. All traces of moisture are removed from the Chlorine and then by a process combining pressure and refrigeration the gaseous Chlorine is liquified and loaded into cylinders, ton containers and tank cars. Under normal atmospheric conditions, Chlorine is a greenish yellow gas with a penetrating

and characteristic odour two and one-half times as heavy as air. Chlorine by itself is non-flammable and non-explosive, although it will support combustion of iron and steel at temperatures in excess of 500°F.

The temperature - vapour pressure relationship is very important. See Figure 1. This characteristic is used in liquifying the product as well as in its withdrawal from cylinders and other containers. At -29°F and at atmospheric pressure Chlorine will liquify. In other words at -29°F it has zero vapour pressure. As the temperature rises there is a sharp increase in vapour pressure. As gas is released the temperature drops and so does pressure. However, if liquid is released, there is no drop in temperature, hence no drop in pressure. This characteristic is very important when dealing with leaks of Liquid Chlorine. This will be discussed in more detail later.

As most of you are no doubt aware, a little Chlorine goes a long way. For example, the least detectable odour is about 3½ ppm. The maximum amount that can be inhaled in one hour without serious effect is about 4 ppm. Fifteen ppm would cause irritation of the throat, 30 ppm will cause serious coughing spells but could probably be breathed up to one-half hour without serious effects. Forty to 60 ppm is considered extremely dangerous for one-half hour exposure. If you were to take a few breaths of air containing 1000 ppm they would very likely be your last.

When an operator knows this, he will not be panicked by his first encounter with a leak, nor will he hesitate to put on his mask before it is too late and he is gasping and sick. In cases of mild throat irritation, drinking milk or cream frequently gives relief. At one time it used to be the practice to keep a bottle of brandy in the first aid room for such treatments, but this was discontinued in favour of milk when it was found that Chlorine leaks were occurring several times a day. First aid treatment will be discussed later.

To meet customers' requirements, Chlorine is generally shipped in cylinders (150# net); ton containers (2000# net) and tank cars of 30 and 55 tons.

All Liquid Chlorine containers are constructed to conform to the specification of the Board of Transport Commissioners for Canada and are tested at prescribed intervals.

Chlorine containers are equipped with one or more Chlorine Institute valves and with a bonnet (or cap) designed to protect the valves from damage or breakage due to knocking or dropping. Letters and numbers are stamped on each cylinder indicating ownership, specification, cylinder number, tare weight and dates of hydrostatic tests. It is illegal to deface these markings in any way.

The contents of any Liquid Chlorine container consist of a liquid phase and a gaseous phase. For reasons of safety, the Board of Transport Commissioners has limited the weight of Chlorine which can be shipped in a container so that when loaded at a temperature of 68°F, Liquid Chlorine will occupy about 88% of the container volume and gaseous Chlorine will occupy the remainder. Cylinders are filled and discharged through a valve with a special bronze body and a monel metal stem. The valve packing is designed especially of graphited asbestos. Valves are equipped with plugs of fusible metal designed to melt at 160°F thereby relieving the pressure and preventing rupture of the cylinder in case of fire or over-heating. This plug must not be tampered with as the full pressure of the gas is behind it regardless of whether the valve is closed or open. See Figure 2.

When the cylinders are returned to the plant, they are degassed (evacuated), thoroughly cleaned, dried and inspected before being refilled for shipment. After cleaning, each cylinder is fitted with a new or thoroughly reconditioned valve. As Chlorine is very cold when filled into cylinders, pressure is low, and in order to detect any leaks before shipping, every cylinder is stored in a warm area for at least 24 hours. This allows the Chlorine to warm up, thereby increasing pressure and indicating if there are any leaks.

MATERIALS OF CONSTRUCTION

When shipped, Chlorine is essentially moisture free. In this dry state and at ordinary temperatures, it does not attack most common metals, zinc and tin being notable exceptions. Most metals, however, will corrode at a rapid rate when exposed to wet Chlorine.

Chlorine combines with water to form Hypochlorous Acid, a powerful oxidizing agent, which few metals can withstand. Leaks occurring in the presence of water or moist air tend to increase rapidly in volume due to the corrosive action of this acid.

Pipe lines for handling dry Chlorine as a liquid or gas should be fabricated from Schedule 80 seamless steel pipe. Copper tubing is satisfactory for the flexible connection at the cylinder. Copper tubing, however, becomes hard and rigid after continued use and should be replaced at least once a year, particularly if Liquid Chlorine is being withdrawn.

Where screwed fittings or joints are used, it is essential that all threads be clean and well formed. Litharge and glycerine or Teflon tape may be used as pipe compound. In making joints, care should be taken to ensure that very little pipe dope is allowed inside the pipe as the oils will be chlorinated to form a gummy substance which may clog in the system. Teflon is recommended because it is inert to Chlorine and is generally used in very small quantities. All cutting oils should be flushed out from the inside of the pipe before it goes into service.

Where large quantities of Chlorine gas are required, it may be necessary to withdraw liquid from the containers and vaporize it in a vaporizer. In this case, precaution should be taken to ensure that at no time is Liquid Chlorine in a line which can be shut off at both ends with a valve, without providing an expansion chamber, to act as a cushion and prevent hydrostatic pressure from rupturing the line. As a general safeguard, Chlorine lines are subjected to a 300 pound hydrostatic pressure test before being put into service.

If a Chlorine line has been opened for repairs or cleaning, moisture may enter and this can cause formation of a brownish syrupy material which is mainly Ferric Chloride. This can cause clogging of chlorinator and pressure reducing valves and other equipment. The best way to prevent its formation is to exclude moisture. During switch-over of cylinders, lines should not be left open to the air - they should be reconnected immediately or plugged off. Corrosion can be kept to a minimum by immediate drying. Clean out of Chlorine lines should be done so as to preclude any residual moisture, following approved procedure.

STORAGE AND HANDLING

Chlorine containers (tonners and cylinders) should be arranged so that they can be removed with a minimum handling of the other containers. Chlorine should not be stored under any of the following conditions:

1. Near combustible or flammable materials such as oil, gasoline, propane, waste, etc.
2. Near the inlet of a ventilating or air conditioning unit.
3. In sub-surface locations.
4. Adjacent to any source of direct heat such as furnace heater, element or radiator.

In addition to the above, in the case of 150 lb. cylinders, the following should be observed:

1. The floor should be even and free of debris.
2. Full cylinders in storage or use should be secured to a wall or post.
3. When moving cylinders, a specially designed 2-wheel hand truck should be used. It is desirable to have a clamping arrangement or safety chain about 2/3 the way up the hand truck.
4. Moving cylinders up or downstairs is not recommended. If this has to be done, the safest way is with a special truck with crawler treads.
5. With experienced personnel a cylinder can be safely moved by rolling it on its bottom edge, provided the floor is not slippery or wet. When rolling a cylinder in this manner, care should be exercised that it does not get out of control and fall, or that the protective bonnet does not screw itself loose.
6. Cylinders should never be lifted by the protective bonnet as it is not designed to support the weight of the cylinders.

Care must be exercised in unloading Chlorine cylinders from truck transports where there is no receiving dock. Cylinders should not be allowed to slide off the end of the truck to ground level. They should be unloaded by means of a hydraulic tail gate or, if this is not available, by a skid. It is also possible for two men working together to ease cylinders from a truck platform to the ground quite safely.

TONNERS

These containers have a gross weight of about 3500 pounds. The recommended method of handling and unloading is by hoist and monorail. A special grab or lifting bar fitted with hooks to fit into the chimes is used for lifting. Storage should be on rails, or in cradles or saddles. Double decking is not recommended unless the lower tier is firmly set in individual saddles. Delivery is generally made by a special truck, although open deck public carriers are sometimes used. Rail cars holding 15 tonners are available also if there are rail facilities.

CONNECTING CYLINDERS

Chlorinating equipment is frequently supplied with a union connection at the end of copper tubing. When this union is connected directly to the outlet of cylinder valve, both union and valve threads will in time become worn or damaged so that a tight connection is difficult to obtain and leaks result. The use of a yoke and an adaptor with the small end fitting into the valve outlet with a small lead washer provides a safe and convenient connection. This equipment is usually provided free of charge by the Chlorine producer. The method for hook-up is illustrated in Figure 2. The hook-up is essentially the same for cylinders and tonners. Procedure is as follows:

1. Secure the cylinder to a building column or a solid upright support. (Not applicable to tonners).
2. Remove the protective bonnet. If threads have become corroded a few sharp raps will usually loosen it. (Not applicable to tonners).
3. Carefully remove the brass outlet cap. As there is a remote possibility of Liquid Chlorine being present under the cap, the use of protective goggles is recommended at this point. Be on the alert for evidence of Chlorine, i.e. odour, or pressure (hissing). Clean out any foreign material in the outlet.

4. Ensure that a new lead washer is in the outlet recess and that it is in the correct position.
5. Place clamp over the valve and insert the adaptor in the outlet recess. Fit the adaptor into the clamp slot, tighten the clamp screw. See Figure 2.
6. Check the packing gland nut to ensure it is finger tight before opening the valve.
7. Opening the valve. The correct procedure for opening the valve is to place the six inch box wrench on the valve stem, stand behind the valve outlet, and while grasping the valve firmly with the left hand, hit the wrench a sharp blow in a counter-clockwise direction with the palm of the right hand. If difficulty is experienced with the bare hand, wear a leather work glove on the right hand. Do not pull or tug at the wrench, as this may bend the valve stem causing it to stick. Under no circumstances should an extension be added to the box wrench as with the extra leverage there is a danger of bending the valve stem, or even shearing it off.

After the valve is opened (one full turn only is necessary) the packing gland nut should be tightened (with spanner or adjustable wrench) and at the same time a check made for any leaks with an Ammonia bottle.

The maximum discharge rate for a single 150# cylinder is about two to three pounds per hour over a 24 hour period. (25 lbs. per hour in the case of a tonner.) A cylinder when discharging gaseous Chlorine is actually refrigerating itself. As the Liquid Chlorine inside the cylinder vaporizes, it takes up heat from the surroundings through the walls of the cylinder. If gas is withdrawn at too high a rate the temperature becomes so low that frost forms on the outside. This acts as an insulation, and reduces the amount of heat the cylinder can get from its surroundings. This condition lowers the Chlorine pressure considerably. This frost may be removed by scraping or by circulating warm air around the cylinder with a small fan. Do Not, under any circumstances, apply hot water or external heat directly to a cylinder to speed up the supply of Chlorine. Remember that the fusible plug is made of metal melting at 160°F. The recommended method of achieving a greater flow of Chlorine gas is to connect two or more cylinders to a manifold. (For much greater demand, liquid discharge may be necessary, and vaporization by means of an evaporator.)

When discharging containers through a common header, care must be taken that all containers are at the same temperature. This applies especially when connecting a new (cold) cylinder to a header. If there is a difference in temperature of the Liquid Chlorine in the connected cylinders, Chlorine will transfer by distillation from the warm to the cool cylinder and may result in its becoming completely filled with Liquid Chlorine. This could be potentially dangerous if the valve on the over-filled cylinder were closed and the Chlorine temperature then raised.

The only reliable method of determining the contents of a cylinder or tonner is by weighing. The pressure in the cylinder is not reliable as it depends upon the temperature and not on the amount of Chlorine in the container. The only sure method of determining the actual contents and to know for sure when it is empty is to weigh the cylinder and check its weight against the tare stamped on the shoulder.

The procedure for closing the cylinder valve is essentially the same as that described for opening it. If the valve does not close tightly on the first attempt it should be opened and closed several times until the proper seating is obtained. A slight click can be heard when the valve is properly seated. Under no circumstances should a hammer or other heavy implement be used to effect a tight closure.

The outlet cap should be replaced, making sure there is a fibre or lead disc in the cap. The protective bonnet should then be screwed back into place.

The final step in returning an empty Chlorine cylinder requires that the portion of the green warning tag below the perforated line be removed. This indicates that the cylinder is empty. If a full or partially full cylinder is being returned, then the green warning tag should be left intact.

TON CONTAINERS

A ton container has two valves very similar to those on the 150# cylinders, the only difference is that these valves do not have fusible plugs. The ton container has three separate fusible plugs in each end. The metal plug in these will melt at 160°F. The positioning of the valves on a ton container is important, when the two valves are in a vertical position, the top valve delivers gaseous Chlorine while the lower valve delivers

Liquid Chlorine. The delivery rate of Chlorine from a ton container, of course, depends on the temperature, but an average flow is about 25# per hour of Chlorine gas at 70°F.

LEAKING CONTAINERS

A recent survey among the Chlorine producers in North America showed that between one and two cylinders in every thousand develop minor leaks after leaving the supplier's plant despite the extreme care which is taken in the cleaning, inspection, filling and valve maintenance and overhaul on all cylinders. Most of the troubles with cylinders arise from two causes, i.e. a turning spindle or leaking fusible plug. A turning spindle is actually a stripped thread on the valve body and results from the fact that the valve stem is made of monel metal whereas the valve body is made of softer bronze. This defect is normally discovered when trying to shut off a cylinder already in service. The simplest way of overcoming this problem is to continue drawing the Chlorine until the cylinder is empty, using the auxiliary valve, or the chlorinator controls. However, under some circumstances, this is not possible and one way of handling the situation is to disconnect the cylinder in such a manner as to include a second valve with the unit disconnected. This valve then acts as a shut-off valve. The cylinder may then be moved to a less hazardous location, and the Chlorine supplier can provide a special emergency device to deal with the situation.

It is also possible to gradually bleed the cylinder to atmosphere or into an alkaline solution (Caustic Soda, Soda Ash or Lime). These solutions will absorb Chlorine at a fairly rapid rate. Another type of leak which occasionally occurs is at a fusible plug. This is usually due to corrosion either internally or from outside. Once again, the supplier can provide a special clamping device which fits over the fusible plug to stop the leak. If the special clamp is not readily available, it is also possible to use the standard adaptor yoke for this purpose, although this should be done only in case of necessity. It should be noted that when an emergency clamp is applied to a cylinder it is now without protection from high internal pressure so that every effort should be made to use this cylinder up as rapidly as possible.

A third possibility for cylinder leak is around the valve packing, which may have become dried out often because of an unusually long time in storage. Some times merely tightening the packing gland nut will suffice, although if

this does not stop the leak, repacking may be necessary by the supplier. It is very important to remember that a leaking Chlorine cylinder be positioned so that gas rather than liquid is escaping. In other words, the cylinder should at all times be kept in a vertical position. With tonners, the leaking valve or relief plug should be kept as high as possible.

A leaking cylinder should never be lowered or dropped into a shallow tank, small stream or pond with the idea of absorbing the Chlorine in the water. This has actually been done in practice. It should be remembered that the solubility of Chlorine in water is very slight, and in the case of a small leak through the fusible plug, the opening is quickly enlarged by external corrosion. Since the cylinder is in a horizontal position, Liquid Chlorine is forced out in such a large quantity that the water is able to absorb only a small percentage, and the remainder bursts out from the surface of the water as a gas. In this situation the refrigeration effect does not come into effect and there is no slowing down of discharge.

Another serious potential hazard can develop if a cylinder is connected directly to a high pressure water main (without a chlorinator), as in the case of chlorinating new water distribution systems. If the water pressure in the main is higher than the Chlorine pressure in the cylinder, water will be forced back into the cylinder. When this occurs, Hypochlorous Acid is formed, heat is generated, often sufficient to melt the fusible plug, thereby releasing Chlorine gas. At the same time rapid corrosion results and instances have occurred where the metal wall of the cylinder has been completely eaten away.

Direct chlorination of water mains is sometimes attempted by a contractor who has little or no experience with Chlorine. This work should be done only by competent water works personnel who, using a chlorinator and with proper operation of valves, can control pressure in the section of line being chlorinated. This should never be allowed to exceed the pressure in the cylinder.

Chlorine lines can be broken through accidental mishandling or Chlorine can be discharged into the chlorinating room by leaving the wrong valve open, etc. When working with Chlorine, care should be taken to follow the prescribed procedures. It is very important that the flexible copper

tube connecting the cylinder to the chlorinator be changed at least once a year, oftener if Liquid Chlorine is being withdrawn. If it is damaged by bending or otherwise, it should be replaced at once.

The chlorinating room itself should be adequately ventilated with a separate emergency ventilating system adequate to provide 30 air changes per hour. The outlet should be located near ground level in the room because Chlorine gas is heavier than air. Ideally, there should be two exits from a room containing Chlorine containers, and the doors should open outwards to facilitate rapid exit. Suitable gas masks should be available in case of emergency. Masks should be located outside the chlorinating room as should the switches for the ventilating system. Respirators of the cannister type do not supply oxygen, but merely absorb the Chlorine present in the air. This type of mask will provide adequate protection in Chlorine concentrations not exceeding one percent by volume. If heavier concentrations are encountered, the use of a self-contained breathing apparatus is essential.

FIRST AID

Prompt treatment of persons exposed to Chlorine is of the utmost importance.

1. Obtain medical assistance as soon as possible.
2. Anyone overcome by or seriously exposed to Chlorine gas should be moved at once to an uncontaminated area. The patient should be placed on his back with head and back elevated. He should be kept warm using blankets if necessary. Rest is essential.

ARTIFICIAL RESPIRATION

If breathing appears to have ceased, artificial respiration should be started immediately using the Neilson, Shaeffer or mouth to mouth methods.

CONTACT WITH THE EYES

If even minute quantities of Liquid Chlorine enter the eyes or if eyes have been exposed to strong concentration of Chlorine gas, they should be flushed immediately with copious quantities of running water for at least 15 minutes. Never

attempt to neutralize with chemicals. The eyelids should be held apart during this period to ensure contact with water with all accessible tissues. If a physician is not immediately available, continue eye irrigations for a second period of 15 minutes.

Even with the best safety precautions, accidents do happen and it is important therefore that people working with or near Chlorine should be familiar with first aid treatment and especially with methods of artificial respiration.

One last word about Chlorine leaks. I hope that what has been said in this talk will serve to dispel any concern which some operators may have had. Since leaks are so infrequent and are generally minor in nature, experienced operators can usually handle them without difficulty. However, to provide for the occasional serious situation, C-I-L maintains emergency equipment at strategic locations and trained personnel can be dispatched on very short notice. An emergency telephone call service is also maintained in Ontario and Quebec which operates on a 24 hour basis to deal with such emergency situations.

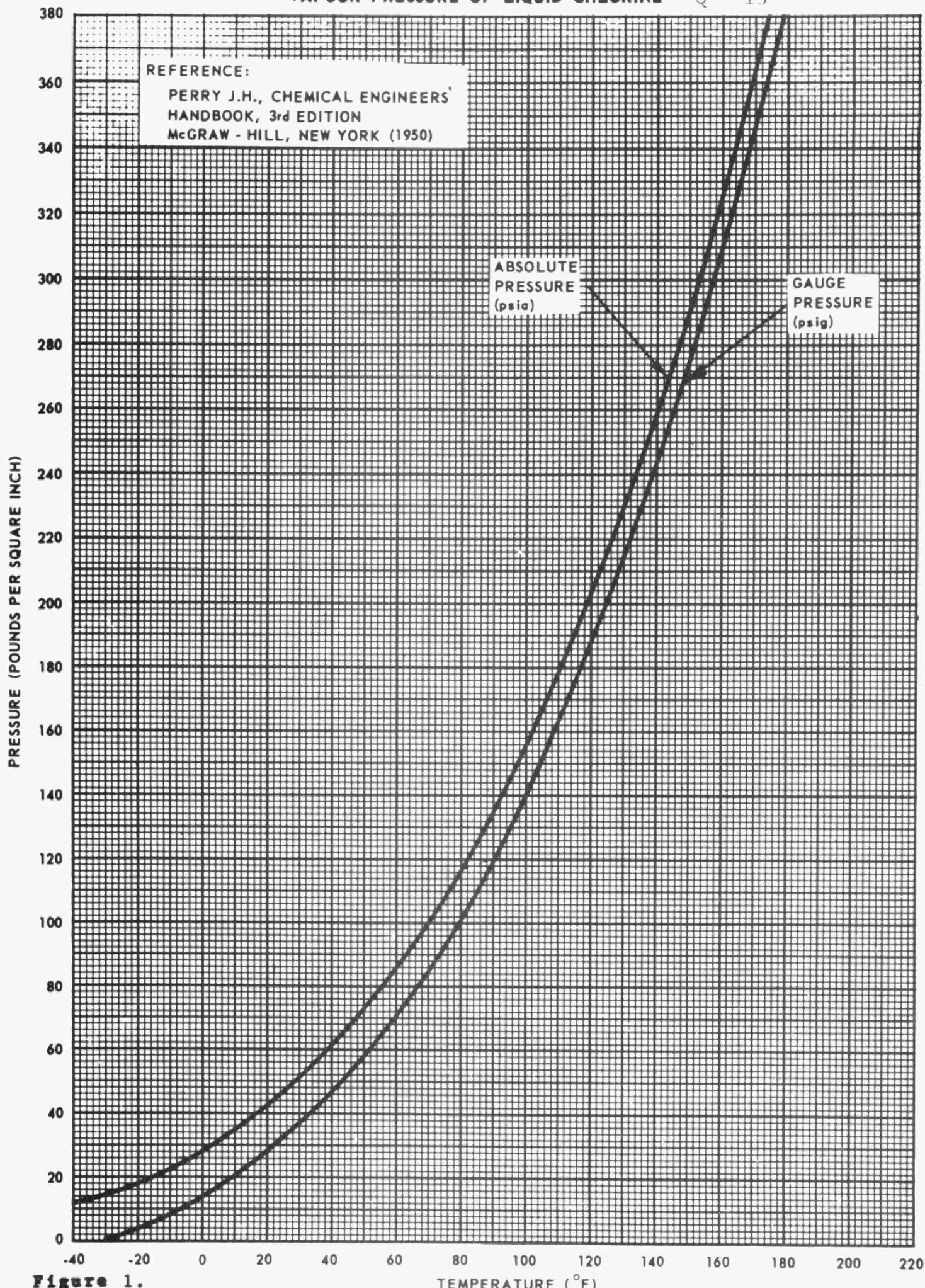


Figure 1.

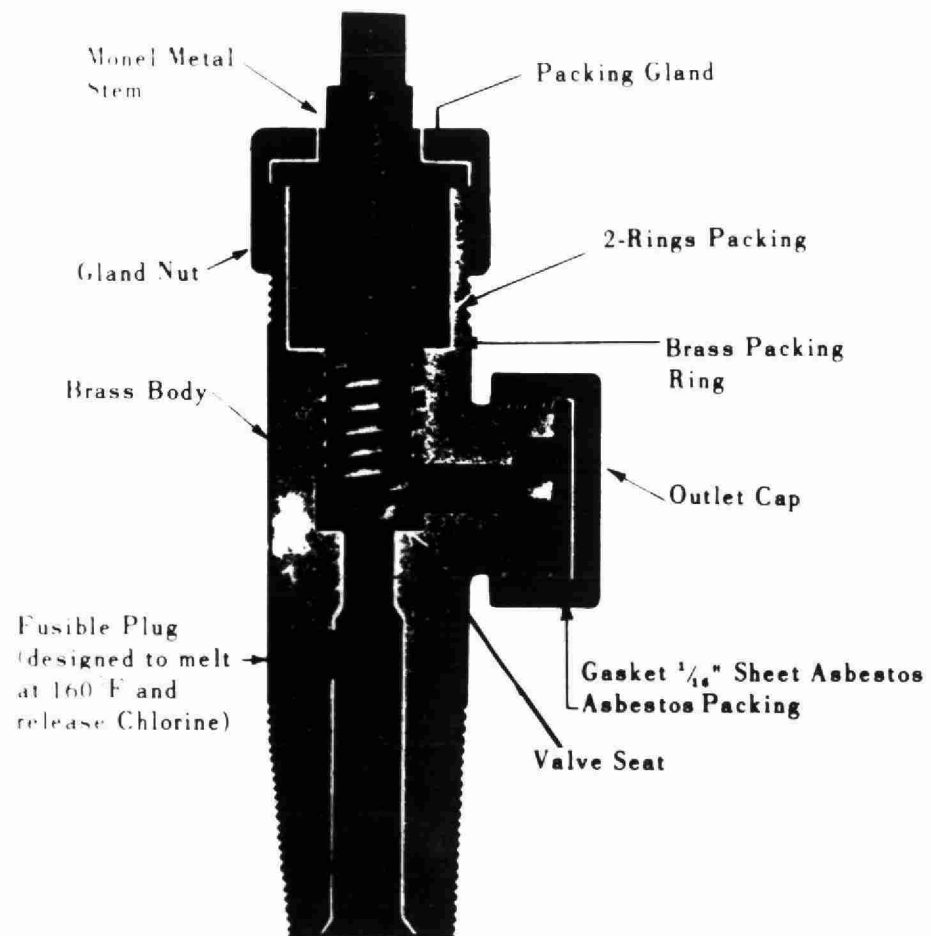


FIGURE 2 Chlorine Cylinder Valve

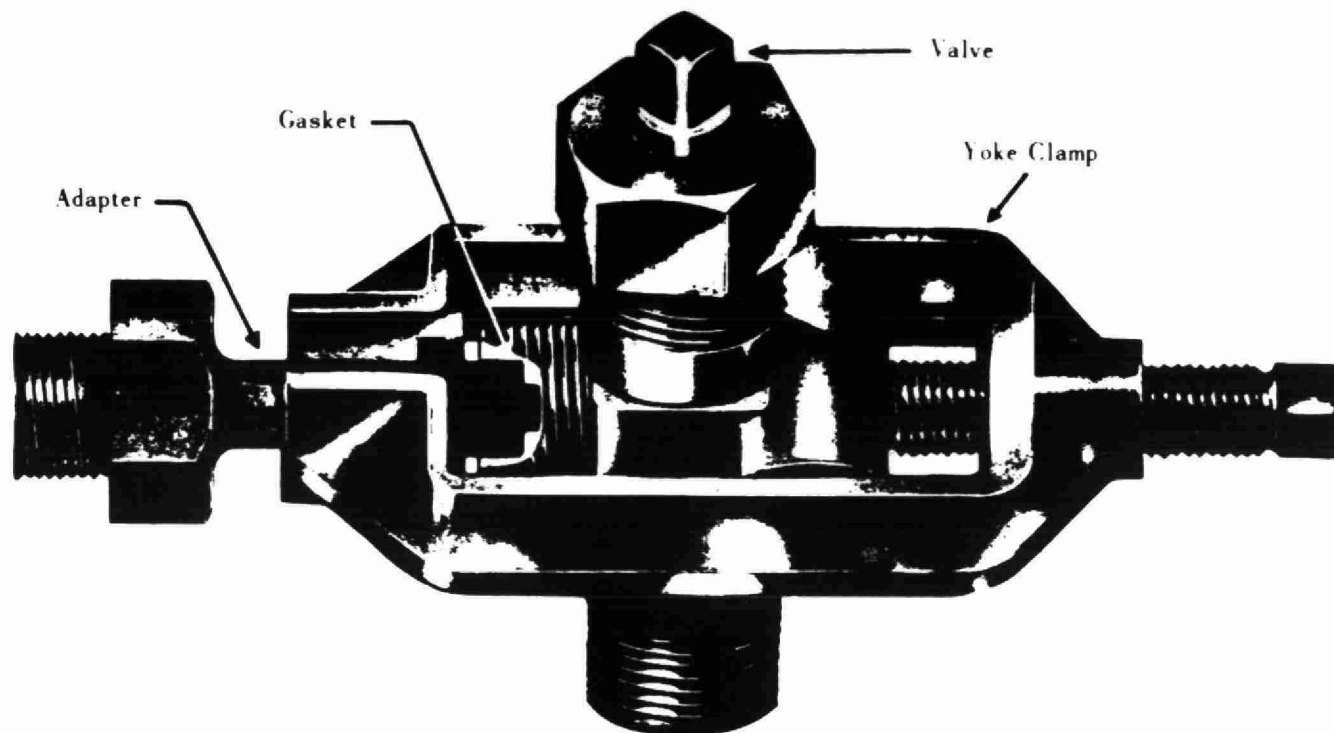


FIGURE 3 Valve Clamp and Adapter Assembly

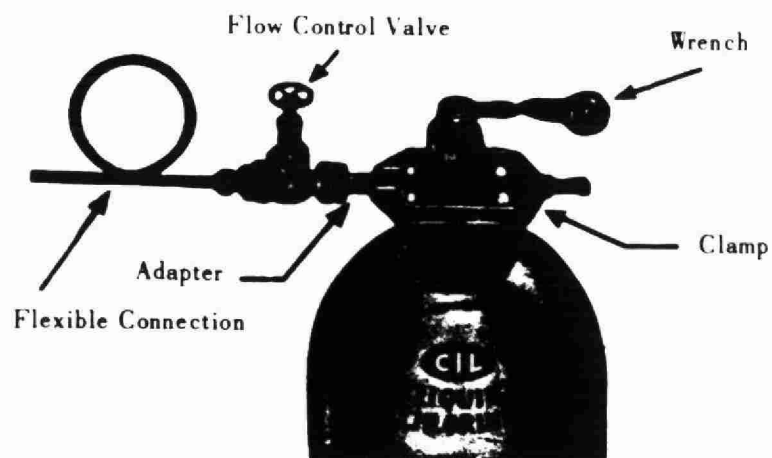


FIGURE 4 Cylinder Connected

AQUATIC LIFE AND WATER SUPPLIES

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PART A

LIMNOLOGICAL ASPECTS OF WATER SUPPLY

INTRODUCTION

The Basic Water Works Course which you have taken included an introduction to the algae, the problems they create in water supplies and some consideration of the methods by which they may be controlled.

Problems created by the presence of algae in raw water supplies are:

- (1) The reduction of filter runs, caused by the coating on the surface of the filters with large numbers of these minute plants, is probably the most common and serious problem that algae create for the waterworks operator.
- (2) Algae are capable of producing tastes and odours that will persist through treatment and cause consumer complaints. Specific species in sufficient numbers cause particular odours (e.g. cucumber, grassy, fishy).
- (3) In reservoirs, algae may grow attached to the walls where they form a heavy mass of material. This algal mass may be alive with crustaceans (e.g. Daphnia) and insect larvae. Occasionally, one of these little animals will come through a water tap and shake the confidence of the consumer in the purity of the supply.

Certain remedial measures which may be adopted include the standard practices of sedimentation, flocculation and prechlorination, as well as the installation of micro-strainers and the use of algicides such as copper sulphate or chlorine in holding basins. Taste and odours may be controlled by either superchlorination, break-point chlorination or where facilities permit, the feeding of activated carbon.

The purpose of the first part of this lecture will be to consider the relevance of limnology as it applies to certain aspects of water supply. It should be recognized that numerous interrelated factors govern the production of algae and other aquatic plants. These include physical, chemical and biological factors as follows:

- (i) physical: temperature, density, diffusion, currents horizontal currents, turbulence, amount of solar radiation, turbidity, ice-cover, etc.
- (ii) chemical: pH, quantity of plant nutrients, oxygen content, presence of trace elements, quantity of non-nutrient ions, the ratio of free CO_2 , HCO_3^- , $\text{CO}_3^{=}$, etc.
- (iii) biological: quantity and nature of enemy organisms, inter-specific and intraspecific co-operation and competition, metabolic rate, life span, rate of reproduction, etc. (from Davis, 1954).

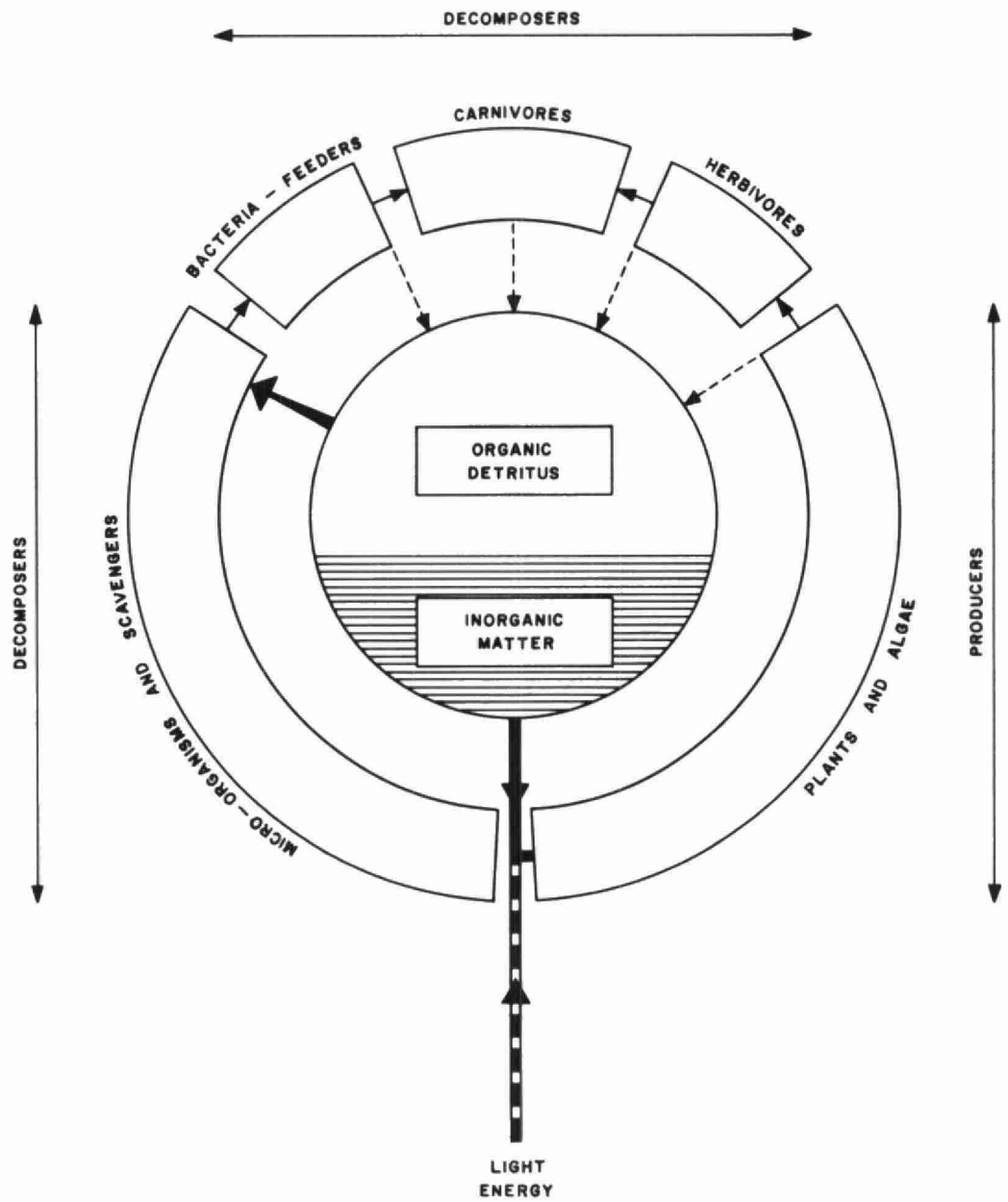
Of these, the more important environmental factors are sunlight, water clarity, nutrients and temperature, and these will be considered in greater detail.

From the second part of the lecture, and during the laboratory period, you will attain a general understanding of:

- (i) the major types of algae which affect water supplies and the nomenclature used in describing algae.
- (ii) the materials and equipment which are necessary for algae identification and enumeration.
- (iii) the procedures used in identifying and counting algae.

BASIC CONCEPTS OF FRESH-WATER ECOLOGY

The understanding of algae as they affect the waterworks operator requires a knowledge of the role of algae in the aquatic environment. Surface waters such as streams, rivers and lakes support active ecosystems in which there is a cyclic interchange between living and the non-living materials. The producer, consumer and decomposer organisms are the three main components in the aquatic ecosystem. The functional relationships within the system are illustrated in Figure 1.



DIAGRAMMATIC REPRESENTATION OF THE AQUATIC ECOSYSTEM
(MODIFIED FROM HAWKES)

FIGURE I

The producers which utilize light energy synthesize organic matter (carbohydrates, fat and proteins) from inorganic sources (carbon dioxide, water and salts of phosphorus, nitrogen, sulphur and potassium). In fresh-water, the producers are represented by the chlorophyll-bearing algae and vascular aquatics. The consumers, in contrast to the algae, cannot manufacture their own food products directly from raw materials. Small consumers, collectively called the zooplankton, obtain their food by eating the phytoplankton. These organisms are in turn eaten by larger consumers (i.e. insects and fish). Waste products and dead consumers, together with decaying algae and aquatic vegetation accumulate to form detritus which is used as food by the decomposers (bacteria, fungi and scavenger animals). These organisms bring about the mineralization of the detritus, thus completing the cycle.

The food chain, therefore, is a closed system into which all nutrients from the watershed enter, are trapped and accumulated. The loss of nutrients through outflowing streams is relatively small. Natural modifications in this ecological system can produce changes which affect the whole order. The process whereby nutrient substances naturally accumulate so that the water becomes progressively more productive is called eutrophication. Essentially, this is the process whereby lakes grow old and eventually die. The rate of eutrophication can be greatly accelerated by increasing the input of nutrient-laden domestic, industrial and agricultural wastes. The resulting imbalances may affect the waterworks operator by modifying the ecological system to the extent that excessive algal growths may develop to impart undesirable tastes and odours to water supplies and interfere with sand-filtration in water treatment plants.

EFFECT OF LIGHT

Plants are photosynthetic. Utilizing radiant energy from the sun, algae can produce carbon dioxide and water, and then assimilate these carbohydrates together with the liberated ammonia and other essentials to produce other algal cells.

Carbon Dioxide + Water $\xrightarrow[\text{and chlorophyll}]{\text{in the presence of sunlight}}$ Starches + Oxygen & Sugars

This is an extremely simplified formula indicating what takes place as plants manufacture their own food. A balanced condition is provided by the fact that animals breathe the oxygen produced by plants in order to metabolize their food and release energy for movement and other bodily activities, at the same time producing carbon dioxide which is essential to the plants.

Any factor which limits photosynthesis would therefore restrict other life processes of the algae. McCombie (1953) reported that turbidity affects phytoplankton production by reducing the quantity and quality of light which is available for photosynthesis. Schenk and Thompson (1965) reported that the high turbidity levels may have had a strong impact by limiting phytoplankton populations at the Toronto Island Filtration Plant from 1923 - 1954. However, it should be cautioned that any attempt to correlate changes in phytoplankton levels with a single physical-environmental factor such as the seasonal variation of turbidity is unjustifiable. It should be emphasized that physical, chemical and biological environmental circumstances acting simultaneously, must be taken into consideration.

EFFECTS OF NUTRIENTS

Of the many chemical constituents which are essential for the production of phytoplankton, attached forms of algae and "higher" aquatic plants, it is generally agreed that nitrogen and phosphorus are of major importance, since they are often in limited supply. While many other trace elements (manganese, iron, cobalt, nickel, etc.) are necessary for plant growth, there is no deficiency of most of these in the aquatic environment. Some algal forms prefer nitrogen in the NH_3 (NH_4) form while others have a preference for the (NO_3^-) form. Additionally, water surfaces are saturated with N_2 . This serves as a source for the nitrogen-fixing algal forms. During the growing season, the various nitrogen forms become somewhat depleted while maximum concentrations are attained in the winter season. Major sources of phosphorus in surface waters are synthetic detergents, industrial wastes, treated sewage and drainage from agricultural lands. Sawyer (1965) concluded in a study of Wisconsin lakes that .30 mg/litre inorganic nitrogen and 0.01 mg/litre soluble phosphorus at the onset of a growing season could produce algal blooms.

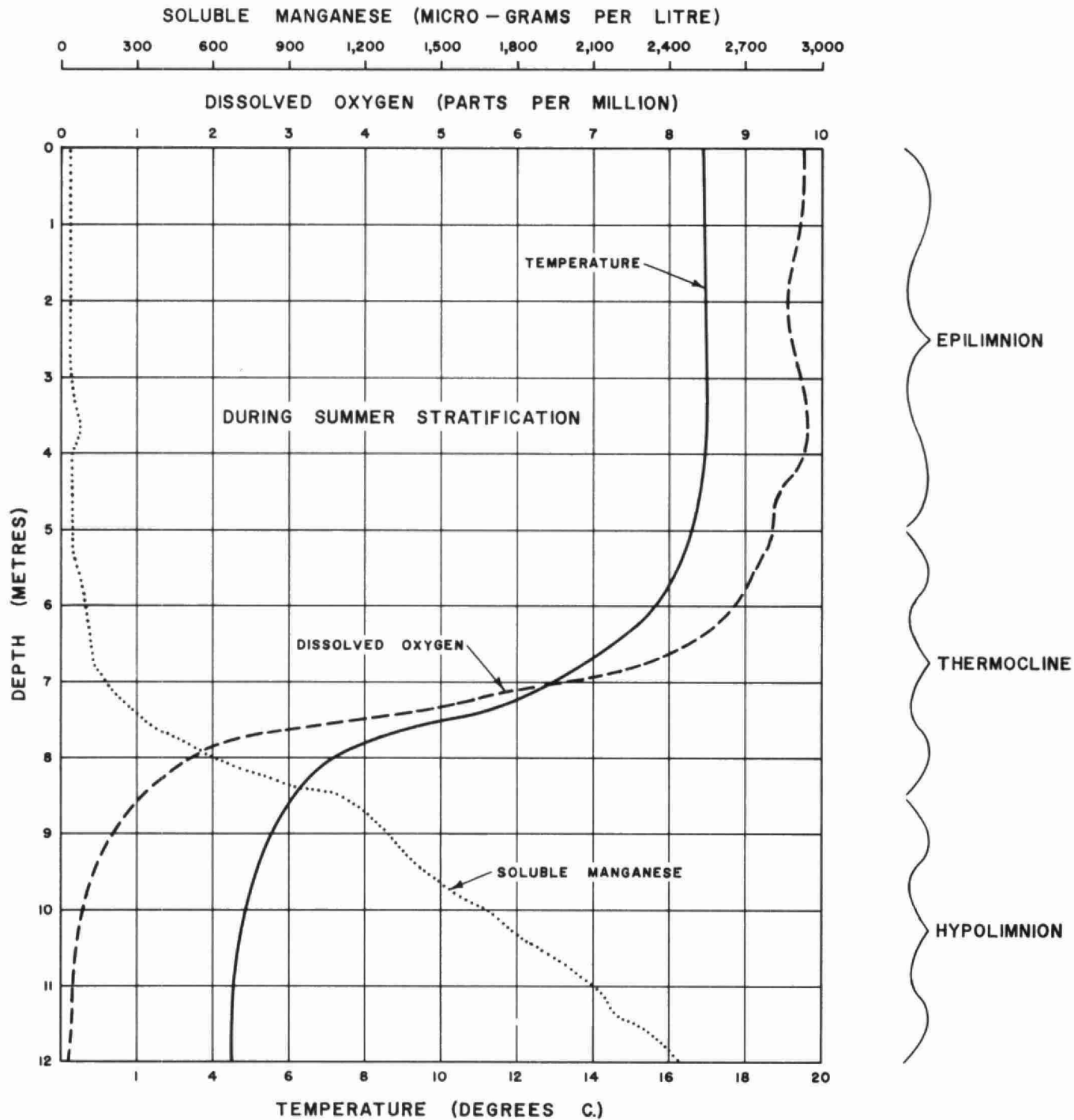
TEMPERATURE EFFECTS

In the late spring and early summer, lake water is warmed by the sun and if the water remains relatively quiet, a temperature gradient is established in which the temperature declines exponentially with depth. (Figure II). Similarly, since warmer water is lighter than colder water, the density of the water will not be the same from the top to the bottom. As the surface-water temperature continues to rise and becomes correspondingly higher, more and more thermal resistance is offered to a mixing by wind action of surface water with the lower heavier water, until a temperature difference of 9°C. or more separates the surface water from that of the underlying water. This situation is called thermal stratification.

The establishment of this stratification establishes three well defined layers of water: the epilimnion, in which the water is essentially uniform; the thermocline, in which there is a rapid drop in temperature per unit of depth, and the hypolimnion in which the temperature from its upper limit to the bottom is nearly uniform.

EFFECT OF THERMAL STRATIFICATION ON LAKES AND RESERVOIRS

The upper layer of a stratified small lake or reservoir is characterized by warm water, good light conditions, an abundance of available oxygen and relatively small quantities of reduced chemical elements (i.e. iron, manganese). Under these conditions, the algae multiply, occasionally to the extent that some may die through exhaustion of nutrients, others from parasites and yet others from the grazing activity of the zooplankton. The dead cells as well as some live cells (some algal forms have a higher specific gravity than that of the surrounding water) sink into the depths. In the hypolimnion, decay of the dead algae and organic matter occurs. As a result, the available oxygen supply is depleted until at the mud-water interface, all of the free oxygen may be used up. An important consequence of this microbial decay is that nutrients (i.e. phosphate-phosphorus, iron silica, and nitrogen) are regenerated but are confined to the hypolimnion because of the barrier presented by the thermocline. Furthermore, the available nutrients cannot be utilized by algal forms because of the reduced light-intensity which is characteristic of the lower regions. However, once the water is mixed either by strong winds or by the fall overturn, these substances are dispersed throughout the entire water mass. In lakes and reservoirs where stratification periods are extensive, it would



MID - SUMMER DEPTH PROFILES OF TEMPERATURE,
DISSOLVED OXYGEN AND TOTAL MANGANESE
IN A SMALL LAKE OR RESERVOIR
(FROM LUND, 1967; AND WELCH, 1951)

FIGURE II

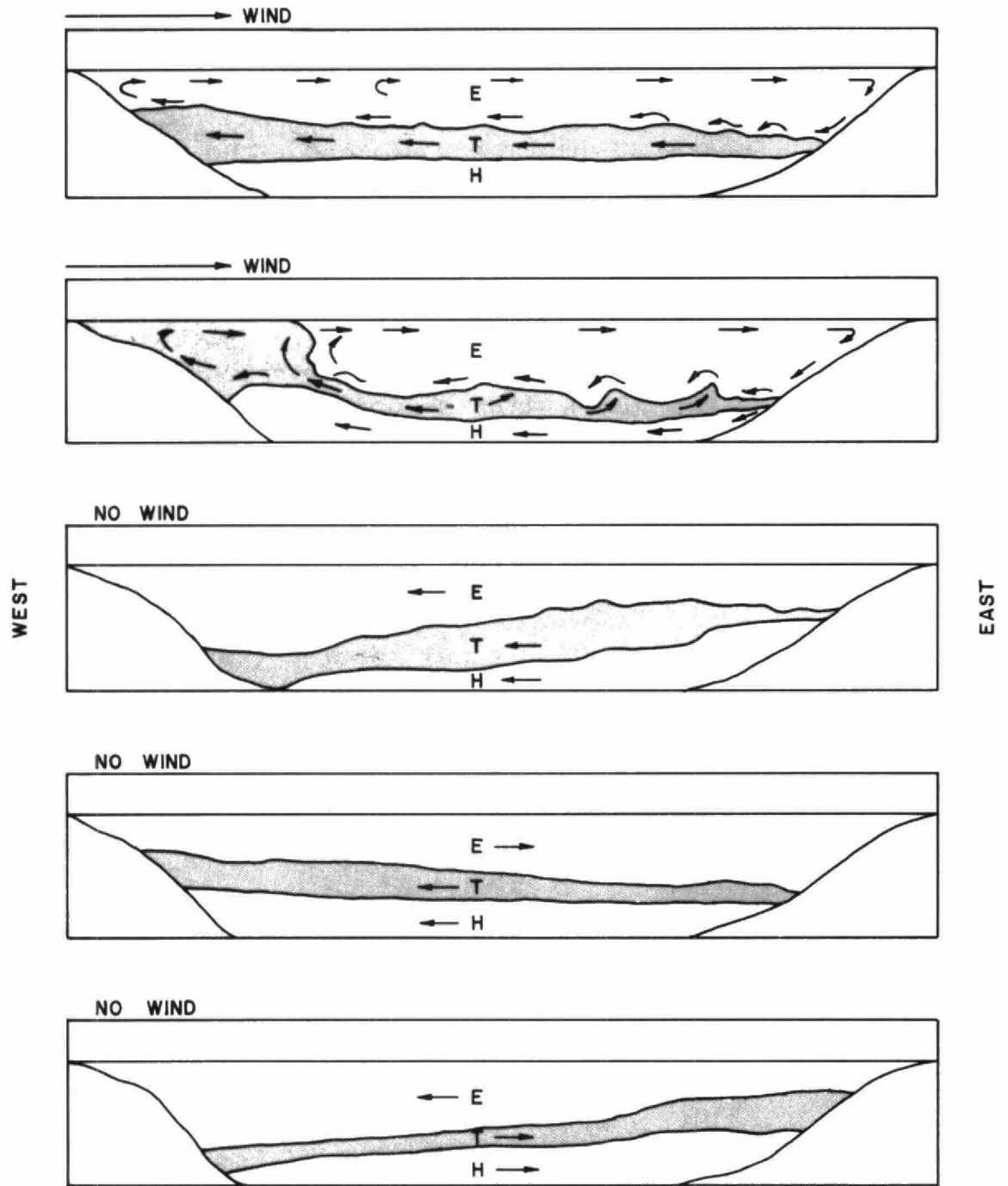
therefore, be undesirable to depend on intakes located near the surface or too close to the bottom. An intake located in the photic zone (i.e. the epilimnion) would draw water containing tremendous quantities of algae, while a supply pipe located in the hypolimnion would contain undesirable sulphides, ferrous iron, manganous manganese, ammonia and a variety of odour producing organic compounds. It would be more sensible, in some instances, to take water from a depth near the lowest part of the epilimnion or in the thermocline zone.

In shallow, fertile lakes with no stratification, consistently high algal populations may prevail. Lakes of this type are seldom suitable as sources of water supply.

EFFECT OF THE INTERNAL SEICHE

Another potential difficulty which influences the quality of the raw water entering an intake (especially in small lakes or reservoirs) is the matter of the internal seiche. The internal seiche is a swinging, see-saw motion of the thermocline which occurs after the wind has blown strongly in one direction. Figure III illustrates the effects of the wind on a stratified body of water. "E" represents the epilimnion, "T" the thermocline and "H" the hypolimnion. When the wind blows from the west to the east, the warm epilimnetic waters move toward the east side of the lake. There is a piling up of the warmer upper waters to the east side of the water mass and a corresponding compensatory shift to windward of the thermocline. The hypolimnion remains undisturbed. If the wind is strong, a breakdown of the stratified layers on the eastern side of the lake occurs. This results in a shift in both the thermocline and the hypolimnion to the western side of the lake. With a stop in the wind, the regular stratification layers are restored but the thermocline continues to swing back and forth with corresponding shifts of the water in the epilimnion and hypolimnion. These movements are illustrated in the last three pictures of Figure III.

Consider now the case where there is an intake location extending into the thermocline from either the eastern or the western shore. With a drop in wind velocity the type of water reaching the supply pipe will differ from time to time according to the regular periodical pattern established by the motion of the thermocline. In a highly productive body of water, an internal seiche could mean that for a period of time, water containing massive



DIAGRAMMATIC REPRESENTATIONS OF AN INTERNAL SEICHE.
FLOW OF WATER IS INDICATED BY DIRECTION OF ARROWS

FIGURE III

growths of algae in the epilimnion would enter the water treatment plant; later in the day, water from the hypolimnion containing the decay - remains of organisms, reduced metallic ions (i.e. iron and manganese), ammonia, undesirable organic compounds, sulphides or even possibly hydrogen sulphide would enter the intake.

In large lakes such as the St. Lawrence Great Lakes, interrupted periods of summer stratification may drastically affect the distribution of polluted water entering a lake. In most instances, the temperature of the in-flowing polluted water is higher than that of lake water and tends to remain at or near the surface of the lake. If a thermocline is present, the polluted water enters and mixes with the upper, warmer epilimnetic waters. If, however, the epilimnion has been displaced by colder upwellings of the hypolimnion, the warm polluted water remains almost as a film on the lake surface. Indeed, pools of polluted waters have been reported (Matheson, 1963) lying immediately over an intake while the deeper, colder hypolimnetic waters were observed entering the water supply intake below.

In the absence of a thermal barrier (during periods of wind stress in the summer months and throughout the major portions of the autumn, winter and spring seasons), the polluted inflowing water disperses throughout all depths of the receiving water. If, for example, the polluted waters were to contain high levels of ammonia, the water treatment plant operator would have to contend with high chlorine demands and possibly the odours of nitrogen trichloride.

CONCLUSIONS

1. Limnological factors should be carefully considered and evaluated when selecting lakes or reservoirs as sources of water supply.
2. Shallow, productive bodies of water are likely to support high algal populations, thus causing notable increases in turbidity as well as influencing palatability and interfering with treatment processes.
3. Careful selection of intake locations is necessary in deeper lakes and reservoirs to minimize chemical and biological problems associated with temperature stratification.

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PART BCHARACTERISTICS OF THE MAJOR TYPES OF ALGAEGENERAL

The algae include all of the microscopic plants which contain chlorophyll but which lack definite stems and leaves as are found in the higher plants. The pigment chlorophyll both in algae and higher plants enables plant cells to convert the radiant energy of the sun into chemical energy, that is, starches and sugars. This process is called photosynthesis and the following simplified formula indicates what takes place:

Carbon dioxide + water + radiant energy $\xrightarrow[\text{(enzyme system)}]{\text{chlorophyll}}$ sugar + oxygen

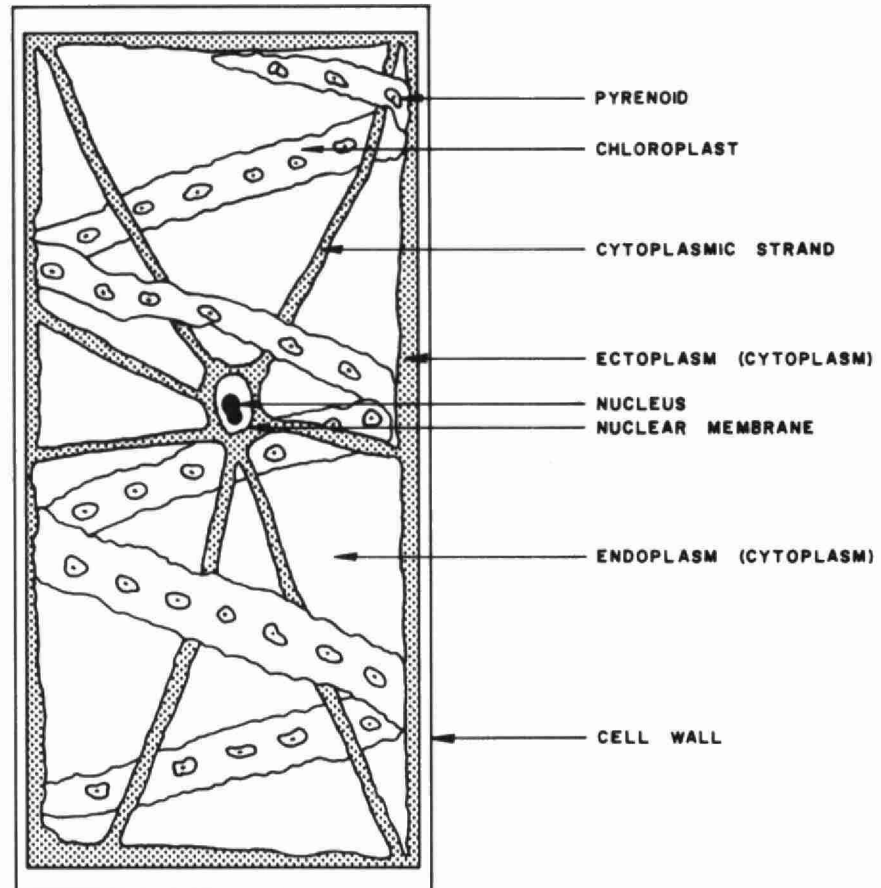
Often, the only significant difference between unicellular plants and animals is whether or not they have chlorophyll and the ability to manufacture their own food.

TERMINOLOGY USED IN THE DESCRIPTION
OF AN ALGAL CELL (Spirogyra)

When viewed under microscope, it becomes apparent that each filament of Spirogyra is made up of cells placed end to end. At first, it would appear that each filament is a flat ribbon, but by focussing up and down, one realizes that the filament has a cylindrical form. Figure I depicts a typical cell of the green alga Spirogyra. (magnification 200x).

Each cell is bounded by a cell wall made of cellulose. Inside the cell wall is the protoplasm, a colourless, gelatinous fluid, common to all plant and animal cells, in which all life activities occur. The protoplasm is divided into two main functional bodies, the nucleus and the cytoplasm. The nucleus is the organization center of the cell and it is suspended centrally by strands of living cytoplasm which radiate from the granular (ectoplasm) layer inside the cell wall. The cytoplasm is divided into the inner viscous endoplasm and the outer granular ectoplasm.

The endoplasm contains the chloroplasts, which in the case of Spirogyra are specialized green bands which extend spirally from one end of the cell to the other. These bands contain the green pigment chlorophyll which is essential for the production of food



DIAGRAMMATIC REPRESENTATION OF AN ALGAL CELL
(SPIROGYRA)

FIGURE I

within the cell through the wonder of photosynthesis. The chloroplasts contain small bodies called pyrenoids. These are the centers of starch formation.

CHARACTERISTICS OF MAJOR TYPES OF ALGAE

Algae, which are important in water supplies, may be placed in four general groups, the blue-green algae, the green algae, the diatoms and the pigmented flagellates.

Blue-Green Algae (Cyanophyceae)

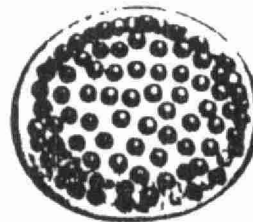
These are the simplest forms of algae and are best described by what they do not have.

- (i) They are the only algae in which the pigments are not localized in definite bodies but dissolved throughout the cell. Blue, red, or other pigments are present in addition to chlorophyll, thus giving the cells a bluish green, yellow, or red colour, at least en masse.
- (ii) No nuclear membrane is apparent
- (iii) No flagella (whip-like appendage to allow movement) is present.
- (iv) Blue-greens tend to achieve nuisance concentrations more frequently in the warm summer months and in the richer waters.
- (v) Vegetative reproduction, in addition to cell division, includes the formation of "hormogones", or short specifically delimited sections of trichomes (filaments).
- (vi) Some species have specialized spores which carry the algae through periods when unfavourable environmental conditions are encountered. Spores of three types are encountered:
 - (a) "Akinetes" are usually long, thick-walled resting spores.
 - (b) "Heterocysts" appear like empty cell walls, but are actually filled with protoplasm and have occasionally been observed to germinate.
 - (c) "Endospores", also called "gonidia or conidia", are formed by repeated division of the protoplast within a green cell wall.

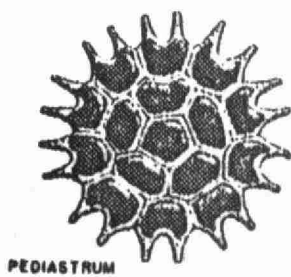
BLUE GREEN ALGAE



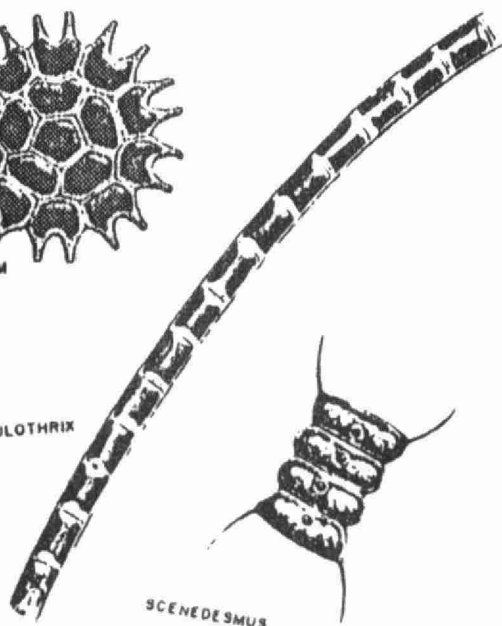
GOMPHOSPHAERIA



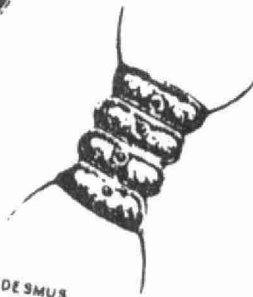
NON-MOTILE GREEN ALGAE (Including Filamentous)



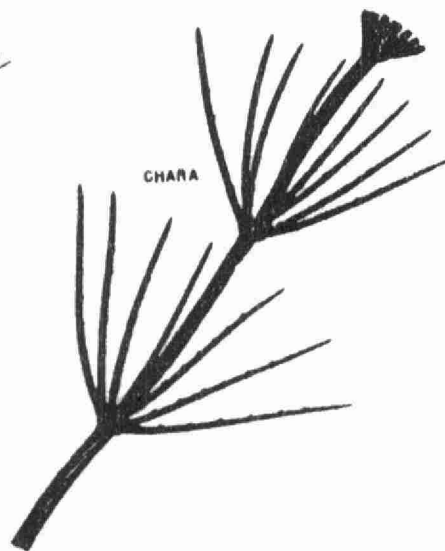
ULOTHRIX



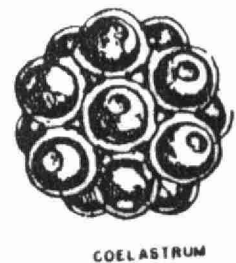
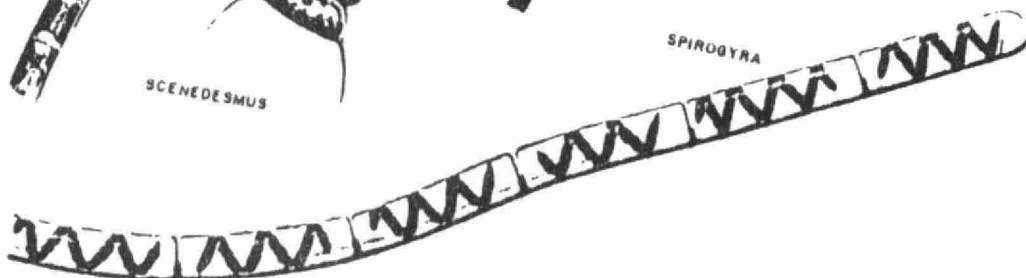
SCENEDESMUS



CHARA



SPIROGYRA



COELASTRUM

(vii) Many species of blue-greens have a definite gelatinous sheath or envelope surrounding each cell, colony or filament.

(viii) There are unicellular, colonial and filamentous species of blue-greens.

Examples: Anabaena (filamentous)
 Anacystis (colonial)
 Oscillatoria (filamentous)
 Chroococcus (single celled or colonial)

Non-Motile Green Algae

The non-motile green algae constitute another heterogeneous assembly of unrelated forms. In this classification system, the yellow-greens and desmids are included with the non-motile greens.

(i) They have a well-defined nucleus.

(ii) They have well organized chloroplasts. These chloroplasts contain chlorophyll. The shape and position of the chloroplast is often distinctive. Pyrenoids are a distinctive characteristic of some members of this group.

(iii) They lack flagella or other locomotor appendages.

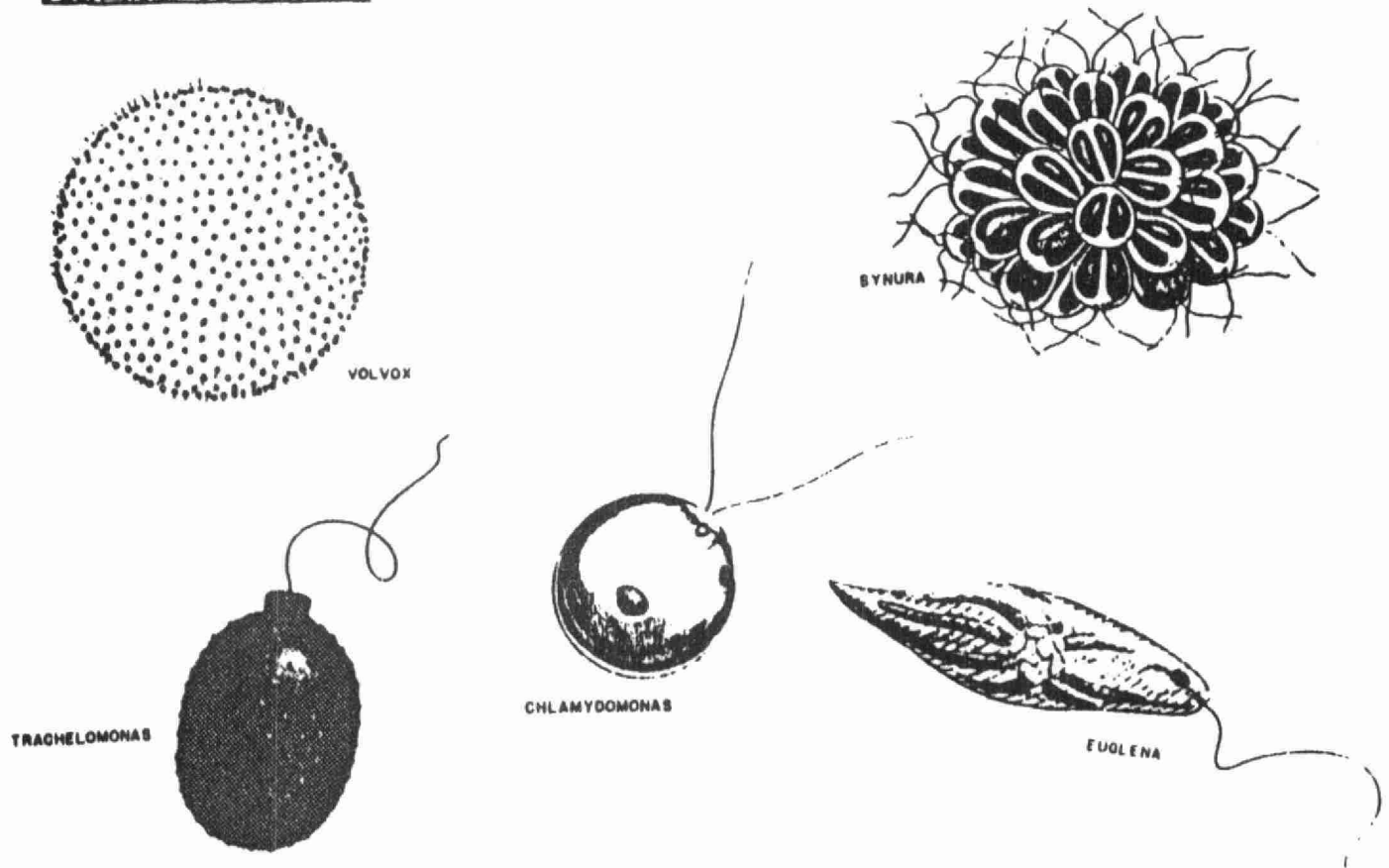
(iv) They have a semi-rigid cell wall.

(v) There is an extreme structural variation among members of this group.

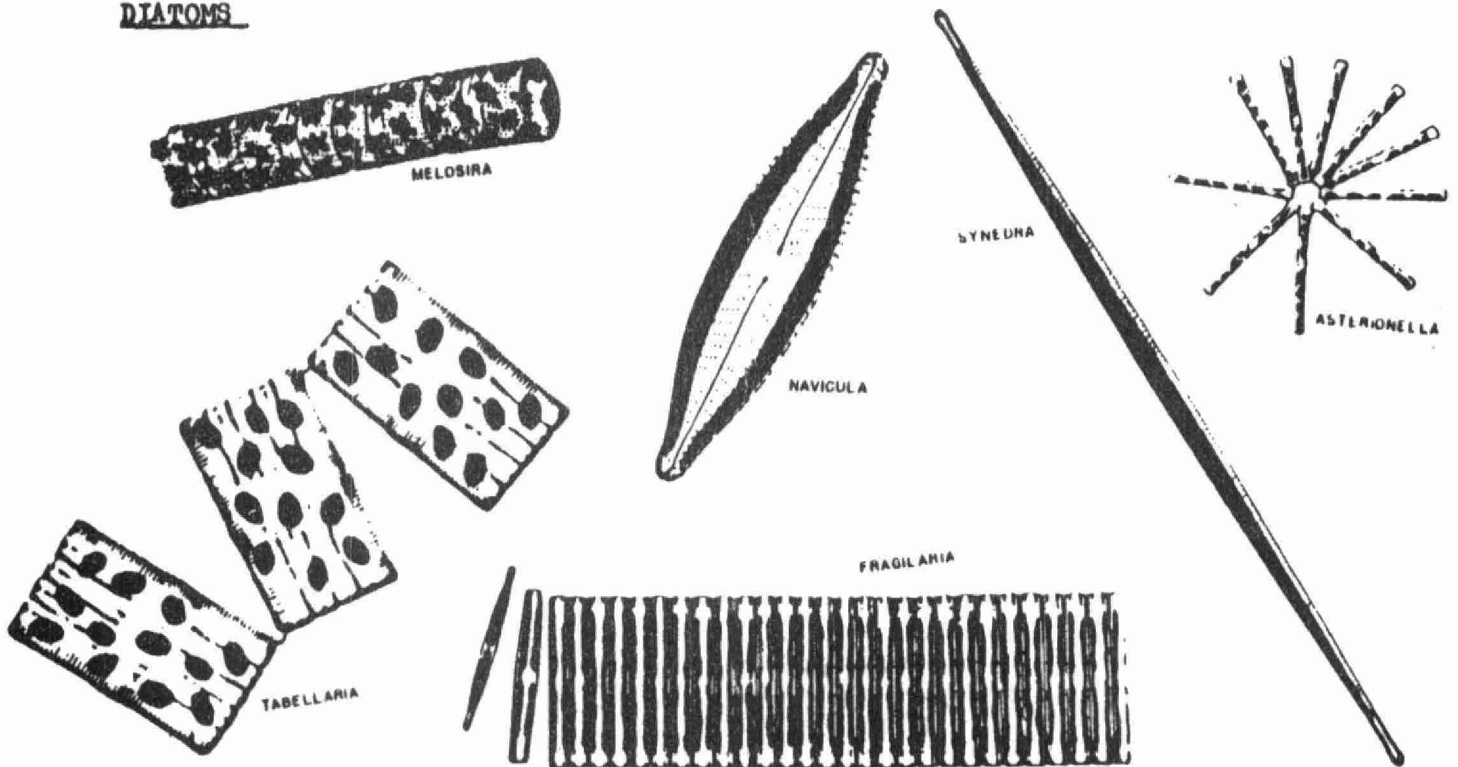
(vi) Some types tend to occur as a general planktonic mass or "bloom", often in combination with two or more species. Some examples are: Sphaerocystis (colonial), Pediastrum (colonial), Scenedesmus (colonial), and the desmid Closterium (single-celled).

(vii) Threadlike (filamentous) algae may form masses or blankets, cutting off light, and reducing water circulation. They also add considerably to the total mass of organic matter. Some examples of this type are: Spirogyra, Cladophora, Oedogonium and Chara.

PIGMENTED FLAGELLATES



DIATOMS



Pigmented Flagellates

The "pigmented flagellates" (in contrast to the non-pigmented or animal-like flagellates) are a heterogeneous collection of motile forms from several different algal groups.

- (i) They have one or more flagella per cell for movement.
- (ii) A thin membrane is usually present surrounding each cell.
- (iii) There is a well-defined nucleus.
- (iv) A light sensitive eye-spot is usually present.
- (v) The chlorophyll is contained in one or more distinctive chloroplasts. In addition to chlorophyll, other pigments may be present in the chloroplasts.
- (vi) Non-motile life history stages may be encountered in some forms.
- (vii) Masses of stored starch called pyrenoid bodies are often conspicuous in certain genera of this group.
- (viii) Some examples are: Euglena (single-celled)
Synura (colonial)
Chlamydomonas (single-celled)
Dinobryon (single-celled or colonial)
Volvox (colonial)

Diatoms (Bacillariophyceae)

- (i) In appearance, they are geometrically regular in shape. The presence of a brownish pigment in addition to chlorophyll gives them a golden to greenish colour.
- (ii) Motile forms move slowly in a distinctive hesitating progression.
- (iii) The most distinctive structural feature is the two-part shell (frustule) composed of silicon dioxide (glass).
 - (a) One part fits inside the other as two halves of a pill box, or a petri dish.

- (b) The surface of these shells are sculptured with minute pits and lines arranged with geometric perfection.
- (c) The view from the side is called the "girdle view", that from above or below, the "valve view".
- (iv) A nucleus is present in all diatom cells.
- (v) There are two general shapes of diatoms, circular (centric) and elongate (pennate). The elongate forms may be motile, the circular ones are not.
- (vi) Diatoms may be single-celled, colonial or filamentous.

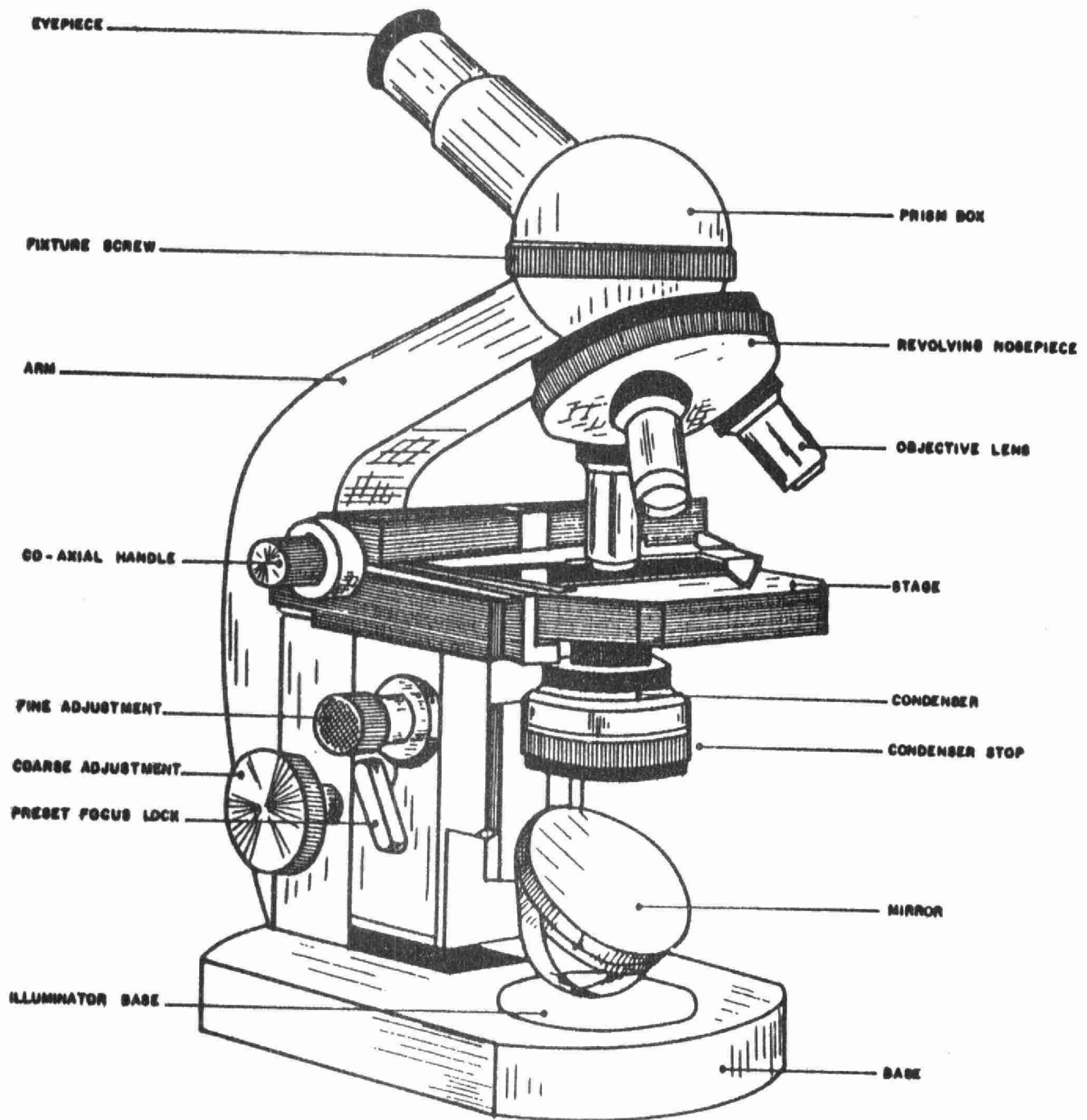
Examples: Navicula (single-celled)
Melosira (filamentous)
Fragilaria (colonial)
Cyclotella (single-celled)

Note: For a summary of the four main algal groups, see page 8 of Palmers Algae in Water Supplies.

ANIMAL LIFE

Occasionally, tiny animals are also encountered in raw water supplies, in addition to algae. These will be single-celled animals which may move by means of flagella or cilia, collectively called protozoa, or multi-celled animals called rotifers which move and capture food by means of two wheel-like crowns of cilia on their heads, or tiny microcrustaceans which are relatives of the freshwater crayfish. Somewhat larger crustaceans, called scuds or amphipods, which may attain lengths in excess of half an inch, may also be encountered. Occasionally, tiny nematodes or roundworms are observed and larval forms of aquatic insects such as bloodworms (midge larvae) are sometimes bothersome. These are only some of the more common small forms of aquatic animal life which may be associated with potable water supplies.

PARTS OF THE MICROSCOPE



EQUIPMENT AND MATERIALS

The most essential and most expensive piece of equipment required for algae counting is the compound microscope.

The compound microscope has two separate lens systems. The one nearest the specimen, called the objective, magnifies the specimen a certain amount. The eyepiece, which is the second lens system, further magnifies the image produced by the objective. Three different objectives are provided on a revolving nosepiece so that high and low magnifications may be obtained. Usually the standard objectives are 10x, 40x and 100x. The names of the various parts of the compound microscope are given on the diagram which has been included.

In addition to the compound microscope, Sedgwick-Rafter counting cells, slides and glassware must be obtained.

PROCEDURES FOR IDENTIFICATION AND ENUMERATION OF ALGAE

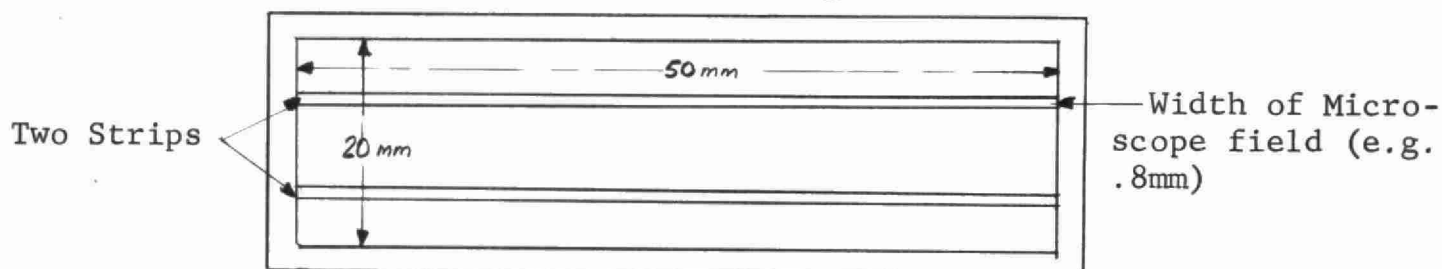
Sometimes it may only be necessary to identify the groups or genera of algae which are present in sources of supply. At other times, it may be desirable to know the numbers of algae, or to determine the total area or volume of algae present. The latter would be useful if shortened filter runs were to become a problem. Most often, counts are made to determine the numbers of clumps (isolated cells plus colonies) which are present.

After a water sample is taken, formaldehyde is added to make up a 3% solution to preserve the organisms present if there is to be any delay before the count is made. Using a pipette, 1 ml. of the sample is placed in a Sedgwick-Rafter counting cell which is covered by a glass coverslip. After waiting a few minutes to let the algae settle, the organisms present are enumerated, using the compound microscope. A Whipple ocular micrometer is placed in the eyepiece of the microscope and, employing a magnification of 200x, the organisms appearing in ten of the fields delimited by the micrometer may be counted. The total is then multiplied by the necessary factor so that the number of organisms per millilitre or litre may be calculated. Sometimes all the organisms in one or more strips across the entire cell are counted and the total multiplied to give the number per millilitre or litre.

Example: How to Calculate Factor for Determining Algae per Millilitre.

Suppose one wishes to count the different species of algae in two strips across the Sedgwick-Rafter cell, these strips having the width of the microscopic field.

The Sedgwick-Rafter Counting Cell



A stage micrometer must be used to determine the width of the microscopic field. Suppose, for example, this width is .8mm.

Interior dimensions of Sedgwick-Rafter cell = 50mm x 20mm x 1mm
(depth)

Therefore, cell volume = 1000mm or
1 millilitre

The volume of water to be checked = 50mm x .8mm x 1 x 2 (no.
= 80 cu.mm. of strips)

Therefore, factor required = $\frac{1000 \text{ cu.mm.}}{80 \text{ cu.mm.}} = 12.5$

The factor 12.5 is used to multiply the total number of each species of algae counted in the two strips to give the number of each per millilitre. The resulting figures may be multiplied by 1000 to give the number of each species per litre.

The ocular micrometer, which is a square grid, inserted in the eye-piece, may be used to measure the length of individual filaments or the areal size of colonial forms. Thus, relative values may be assigned to filaments and colonies rather than assigning the same values regardless of size.

It is quite simple to learn how to use a microscope properly. The counting cell is placed under the low power objective and the coarse adjustment knob is turned until the objective is about an eighth of an inch from the coverslip.

One should then look into the eyepiece, and with microscopes having a moveable stage, the stage should be lowered with the coarse adjustment knob until the organisms on the bottom of the counting cell come clearly into view. Fine focusing can be achieved with the fine adjustment knob. When the high powered objectives are used, the coarse adjustment cannot be turned slowly enough as the object comes close to being in focus, and one must use the fine adjustment to obtain a sharp image. If there is too much light and resulting glare, the condenser diaphragm should be turned under the microscope stage to reduce the size of the diaphragm opening. It is essential to remember that the image formed by the compound microscope is inverted; the object is seen upside down and reversed so that the right side is at the left.

OPERATION AND MAINTENANCE OF MUNICIPAL WELLS

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INTRODUCTION

About 60 percent of the water works systems in Ontario obtain their water supplies from wells. Wells therefore play a significant role in the water supply field and like any surface water supply, require certain attention if the maximum benefits are to be derived at the most economical means.

The operation and maintenance of a well are often taken lightly by well owners. The installation and maintenance of the pumping equipment is often given more attention because at least some of its working parts are above ground. A well, because it is underground and out of sight, is often overlooked until trouble occurs and demands immediate and perhaps costly action.

Certain general operating procedures exist for wells as for surface water supplies and if followed, will tell the well owners how efficiently the well and pump are operating, how the water level in the ground water source is behaving, how adequate the supply is, and when action is required to rehabilitate the existing well or to install additional supplies into the system. Water wells cannot be expected to last forever, but systematic operation and preventative maintenance, properly worked out for the specific conditions in a locality, will assure a dependable source of supply and improve the overall performance and increase the life of the wells.

To properly operate or maintain a water well a person should have a working knowledge of the following:

1. the methods and materials used in constructing wells.
2. the local water-bearing formations or aquifers and their hydrologic characteristics.
3. the quality of ground water and its effect on well materials.
4. the causes of well troubles, how to recognize them and what can be done to eliminate them.

WELL CONSTRUCTION

All water wells fall into two classes: overburden wells which obtain water from aquifers in unconsolidated formations lying above the bedrock and rock wells which obtain water from aquifers in the bedrock. The methods of construction differ greatly for the two types of wells. Consequently, the problems connected with well operation and maintenance are partially dependent on the type in use.

Overburden wells must depend on some kind of screening device below the casing to let the water, which flows through the pore spaces in the aquifer, into the wells. Well screens are intended to serve two purposes; to let water into the well with a minimum of friction loss and at the same time to hold back the particles of the formation in which the screen is placed. Many overburden well failures are due to poorly designed or inefficient well screens.

In contrast, rock wells depend on openings in the consolidated formations for their water supply and the extension of the drilled hole through the water-bearing zone is usually the only procedure needed. The number of openings intersected depends largely on chance and the amount of water yielded from different wells in the same formation may vary widely.

HYDROLOGY

Regardless of the type of well, in the final analysis, several factors determine how much water can be obtained.

Over the long-term outlook, the yield depends on the replenishment of water by rainfall or other recharge to the aquifer. When more than the amount of recharge is withdrawn, there is a continuing decline in water level, which eventually could result in failure of the well.

The rate at which water can be withdrawn from a well annually in a particular aquifer depends on the hydrologic characteristics of the aquifer and the efficiency of the well. These can be determined only by means of pumping tests.

The measurements that are made in testing wells include the volume of water pumped per minute or per hour, the depth to the static level before pumping is started, the depth to the pumping level at one or more constant rates of pumpage, the recovery of the water level after pumping is stopped, and the length of time the well is pumped at each rate during the testing procedure. These measurements are particularly important as they not only furnish the basis for estimating the perennial yield of the well but they also determine the proper setting of the pump bowls.

WATER QUALITY

The chemical constituents of ground water depend largely on the chemical nature of the formation itself, the rate of flow and the temperature. As a result the quality varies widely. The ideal is a chemically balanced water but usually ground water is either corrosive or scale forming. Either case will add to the deterioration rate of the well, unless methods and materials are employed which tend to retard their effects.

PROBABLE CAUSES OF WELL FAILURE

From the preceding we can see that, although there are others, the three most common types of well failures are:

1. Failures due to faulty initial well design and construction. Such failures are indicated by difficulty in setting and operating pumping equipment, by partial or total collapse of the well casing, or by the pumping of large quantities of sand.

2. Failures due to pumping the ground-water reservoir at rates in excess of the rate of replenishment. Such failures are indicated by a continual lowering of the static level and pumping level in the well, as well as in the aquifer adjacent to it and are accompanied by a decrease in production.
3. Failures due to screens becoming clogged, incrustated or corroded in overburden wells and due to the openings in rock formations becoming plugged or incrustated. Such failures are indicated by increased drawdowns in the well accompanied by a falling off of capacity, while the water level in the adjacent formation remains the same, or by the continuous pumping of fine sand.

REMEDYING WELL TROUBLES

Well failures due to faulty design and construction are in the minority. Such failures are caused mostly by the use of cheap materials, poor construction methods, improperly designed screens as far as strength, diameter and slot size are concerned, poorly designed and placed gravel packs and poorly plugged bottoms. These failures usually give little or no warning.

Difficulties in setting and operating pumping equipment arise when casing diameters are too small to accommodate adequately the pump with the result that the slightest misalignment of the casing causes undue stresses in the pump column. As a general rule inside diameters of casings should be at least 2 inches larger than the pump.

Collapses of casings, while rare, are possible whenever casing is used that is not strong enough to withstand the vertical and lateral pressures that it will be subjected to under pumping. In actual practice collapses are prevented by using casing that is usually many times stronger than need be.

Poorly designed screens are the cause of many well failures. Slot sizes, lengths and diameters of screens should be selected with reference to the grain sizes in the aquifer and the designed capacity of the well. Slot sizes too small

for the formation are often specified in order to save development work. Improper selection of screens often results in excessive drawdown in the well because of increased friction losses.

Tremendous pressures are placed on bottom plugs and if the plugs are not carefully placed these pressures gradually loosen them, resulting in leakage of sand into the well.

Generally, failures of the kind I have just described occur in wells where it was desired to reduce costs to a minimum. This is false economy of the worst kind.

Well failures due to pumping the ground-water reservoir at rates in excess of the rate of replenishment are not infrequent and are usually the result of a general lack of knowledge on how to rate the yield of a well.

When a well is pumped, water is taken out of the underground reservoir by the reduction of pressure at the well, thus causing a flow towards the well. If the formation is not pumped excessively, the pumping level will eventually reach a point where replenishment to the aquifer is equal to withdrawal and equilibrium is established. On the other hand, if the formation is pumped excessively, the pumping level will continue to go down as long as the reservoir will supply water and will finally result in failure of the well. In actual cases what happens is that the pumping level gradually lowers in a well until it is too low for the pump to operate.

Overpumping results in low pumping levels thereby increasing operating costs as well as increasing the rate of clogging of well screens. Such conditions can be discovered only by means of performance records for the well and can be corrected only by regulating the pumping rate to conform with the rate of replenishment. This principle of well operation controls not only the quantity that can be safely pumped from one well but also the quantity of water that can be developed from a group of wells in a given area.

In order to avoid failures due to overpumping, the performance of all wells should be carefully recorded regularly, so that it can be determined when the maximum safe pumping rate has been reached.

Well failures due to inlet openings or screens becoming incrustated or corroded are probably in the majority.

Corrosion and incrustation are phenomena which are basically due to chemical changes in the water being pumped due to a reduction of pressure and an increase in the velocity of the water in the aquifer and entering the well. Corrosion of metals is a chemical action set up by the environment to which they are exposed, resulting in deterioration or eating away of the metal. Incrustation, on the other hand, is an accumulation of mineral salts or other extraneous matter in and immediately behind the openings in a well screen.

Corrosion of well screens is not as prevalent as incrustation but it is more harmful, because the metal itself is destroyed. Without considerable experience and some investigation it is difficult to recognize the type of corrosion involved, as there are at least six different forms.

The rate of corrosion is dependent on a number of factors, one of them being the rate of movement of water over the surface being corroded. To offset corrosion in wells and pumps, two general methods of approach are common; one is the use of protective coatings and the other the use of highly corrosion-resistant metals. Neither method has proved to be a wholly effective safeguard against corrosion.

Incrustation of well casings, openings, and screens is a deposition of materials in and around all metal parts of wells and pumps. These accumulations are made up largely of the bicarbonates and sulphates of calcium, magnesium, sodium and iron with a variety of lesser minerals. Other agents contributing to incrustation are iron bacteria and slime-forming organisms.

No way has yet been found to prevent or entirely remove these accumulations in and around well screens and pumps. Steps can be taken to retard such accumulations. One way is to design screens so that friction losses in the water passing through the screens and the aquifer are kept as small as possible. Another step is to lower the rate of pumping and increase the period of pumping. This will be effective if drawdown is materially reduced. Some well owners find it more economical to operate a large number of properly spaced wells, with less drawdown, than to try to obtain their requirements from a few wells having excessive drawdowns.

If it is suspected that incrustation is present, periodic cleaning of wells and pumps should be a regular item of maintenance if costly rehabilitation or well replacement is to be avoided.

The nature, construction and hydrologic characteristics of rock wells are different from overburden wells and the maintenance and operation of them is somewhat different. Mostly it is a question of good judgement coupled with experience.

Mechanically there is very little that can be done to maintain a deep rock well supply. Sometimes it is necessary to go into such wells for the purpose of keeping the fractures or solution channels open. If the drill hole has become partially filled with sand or rock debris it is cleaned out by bailing. Chemical treatment and surging are helpful in dissolving certain kinds of accumulations in the fractures.

One of the main causes of failure in rock wells is the lowering of the water level to a point at which it is no longer economical to pump the well. This, of course, can be avoided by the regulation of pumping rates to the amount that can be safely withdrawn annually.

RECORDS TO BE KEPT

From the foregoing we can see that the keeping of accurate periodic records of pumpages, static levels, and pumping levels in wells would be essential and invaluable in any emergency which may arise. Such information would also be helpful in anticipating shortages due to increased demands or failure of wells and pumping equipment. If records are to be kept it is essential that the readings be taken accurately by the use of approved measuring devices and standard methods.

METHODS OF MEASURING WATER LEVELS

For measuring a water level the common unit is the foot, though it is sometimes necessary to take the readings in other units and convert them to feet for recording.

To make readings to any water level the simplest way is to use a weighted, chalked steel tape. Readings taken in this manner are usually the most accurate. However, it is often difficult to get such a line down a well because of the manner in which the pump is installed. Portable electric depth gauges also fall into this category.

A satisfactory method and probably the most common is to use an airline and pressure gauge. This apparatus is inexpensive, easy to install and remove, and is generally accepted for all but the most accurate tests. It consists essentially of small diameter pipe or tubing extending from the surface to a point below the maximum expected pumping level. The line is usually fastened to the pump column with the bottom being at least 10 feet above the intake of the pump because high velocities will affect the readings. On top of the tubing or pipe is an ordinary altitude or pressure gauge to which is attached an ordinary tire pump or air pressure hose if available. In order to make accurate readings with this device it is necessary that the airline be air tight from one end to the other and that the exact vertical distance between the centre of the gauge and the open end of the airline be determined.

METHOD OF USING AIRLINE

When the airline is installed it is full of water up to the level of the water in the well. When air is forced into the line, it creates a pressure which forces all of the water out of the lower end filling the line with air. If more air is then pumped in, air instead of water is expelled and it is not possible to increase the pressure further. The head of water above the lower end of the airline maintains this pressure, and the gauge shows the pressure or head required to balance the water pressure. If such a gauge is graduated in feet, it will register directly the amount of the airline that is submerged. The submerged portion is subtracted from the total length of the line to give the depth to the water level. To simplify the readings even further, a new type of gauge is now available and in common use today. It is simply a pressure gauge graduated in feet with the scale reversed. All that is required is that the needle pointer be set under zero pressure at the number corresponding to the length of the airline. Under pressure the pointer will indicate the exact depth to the water level.

For maintenance purposes water level readings should be taken daily, especially the pumping level. Static levels should be taken at least once a week. In each case the reading should be taken after the longest period of pumping or recovery, respectively.

METHODS OF MEASURING QUANTITY

While the measurements of water levels are being made it is just as important that measurements of the quantity being pumped at those levels be made. Generally flow meters are standard equipment in most pumphouses. If they are not they should be. Records should include a daily reading of the total daily flow. It is periodically found necessary to retest a well for production especially if it is suspected that production has fallen off. During retesting wells for production, it is important that constant rates of flow be maintained. This can usually be done most accurately if the flow is measured while the water is being pumped to waste at the pumphouse. There are numerous methods for measuring the flow, some of these being Venturi meters, Pitot tubes, current meters, orifices, flow gauges, and weirs. Devices of this kind are easy to install but they must be installed correctly and the readings taken carefully.

The need for a compact, easily installed, and reasonably accurate measuring device has led to the general acceptance and use of the circular-orifice meter. The device consists of a sharp-edged circular orifice at the end of a horizontal discharge pipe. The orifice is from $1/2$ to $3/4$ of the diameter of the pipe. The pipe must be smooth internally and free of any obstructions for a distance of at least 6 feet from the orifice. The discharge pipe is tapped on the side to provide a connection for a rubber or plastic hose or glass tube so that the pressure or head in the discharge pipe can be measured at a distance of 2 feet from the orifice. The discharge pipe must be horizontal and the stream must fall free from the orifice. The orifice must be vertical and centred in the discharge pipe. The combination of pipe and orifice diameters for a given test should be such that the head measurement will be at least 3 times the diameter of the orifice.

Tables are available which give the discharge in gallons per minute through circular-orifice meters of various sizes corresponding to the heads observed in the manometer tube.

PROTECTIVE MEASURES

In operating wells there are a number of things that should be done to maintain and protect a ground-water supply. Some of these are as follows:

1. All wells should be located and constructed by men experienced in this type of work and who understand the occurrence and movement of ground water in the various water-bearing formations. Improper construction and improper rating have been the cause of many well failures.
2. All wells should be located so that they will not interfere seriously with other wells operating in the vicinity.
3. All wells and pumping equipment should be constructed of materials that will resist the corrosive and erosive action of the water being pumped, as well as the action of chemicals which may be used to remove accumulations.
4. All wells should be operated at their most economical and efficient point by equipping them with pumps properly designed for the characteristics of each well.
5. Water samples should be collected and analyzed regularly and periodically for chemical and bacteriological quality. The quality of the water may change slowly as the well is used. It may improve or it may get worse, depending on the geologic and hydrologic conditions of the area. Chemical analysis may indicate whether incrustation likely is occurring and what steps should be taken to prevent serious plugging of the well.
6. Last and possibly most important is the keeping of good records of well operation. Accurate records of the history of the well undoubtedly provide the soundest basis for deciding just what preventive maintenance procedures would likely be most worthwhile.

7. A carefully conducted pumping test carried out periodically in the prescribed manner is also advisable. Such tests, when interpreted in the light of good data on the past history and known changes, assist greatly in developing practical maintenance procedures.

CONCLUSIONS

In conclusion we can see that good operation and maintenance of a well supply is as important as that of pumping equipment and are a responsibility of the owner which should not be taken lightly or ignored completely. If it is expected to get the most out of a well supply, good construction methods should be employed, proper pumping equipment installed, and adequate supervision provided.

Supervision is the responsibility of the water-well operator; if this is to be done properly he should inform himself regarding the wells under his care.

CORROSION CONTROL

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WHAT IS CORROSION?

In the waterworks industry, CORROSION is defined as the action of water in dissolving iron, steel, copper, zinc and other metals that are used in the manufacture of conduits, pipes, watermains and other equipment used in the production and distribution of water.

WHY DOES CORROSION TAKE PLACE?

With the exception of a few noble metals such as gold, silver and platinum, most metals are very rarely found in nature in the pure form. Many are found at some stage of oxidation or combined chemically with other elements in the form of an ore. Nearly all of the metals used in the manufacture of waterworks equipment are produced by reduction or smelting of ores. They are considered less noble because they are unstable when exposed to the forces of nature. They have a strong tendency to corrode. In other words, they will revert to a more stable form similar to that of the original ore if left unprotected such as in the case of rusting or the formation of iron oxide in the corrosion of iron.

IMPORTANCE OF CORROSION

Corrosion costs money because it helps to wear out and destroy watermains and other necessary equipment used in the waterworks industry. Studies have shown that financial losses due to corrosion could run as high as 10 percent of the gross revenues from an average municipal waterworks plant.

Corrosion causes many nuisance problems such as discolorations and adverse taste and odour conditions which may affect the water quality in the distribution system.

THEORY OF CORROSION

Corrosion of metal in water is a complex electro-chemical reaction. Perhaps, the easiest way to explain and understand corrosion is to examine what happens in a dry cell. This is a simple application of the corrosion principle in action.

Basically, a dry cell is made up of three essential parts: anode, cathode and electrolyte (Figure 1). The anode is the negative terminal of the dry cell and it is attached to a zinc container. The cathode is the positive terminal consisting of a carbon rod (electrode) at the centre of the dry cell. The electrolyte is any liquid or solution that contains ions and has the ability to carry an electrical charge. In a dry cell, the electrolyte consisting of a mixture of ammonium chloride, water and some other chemical agents is placed in a zinc container. The carbon electrode is suspended in the electrolyte at the centre of the cell.

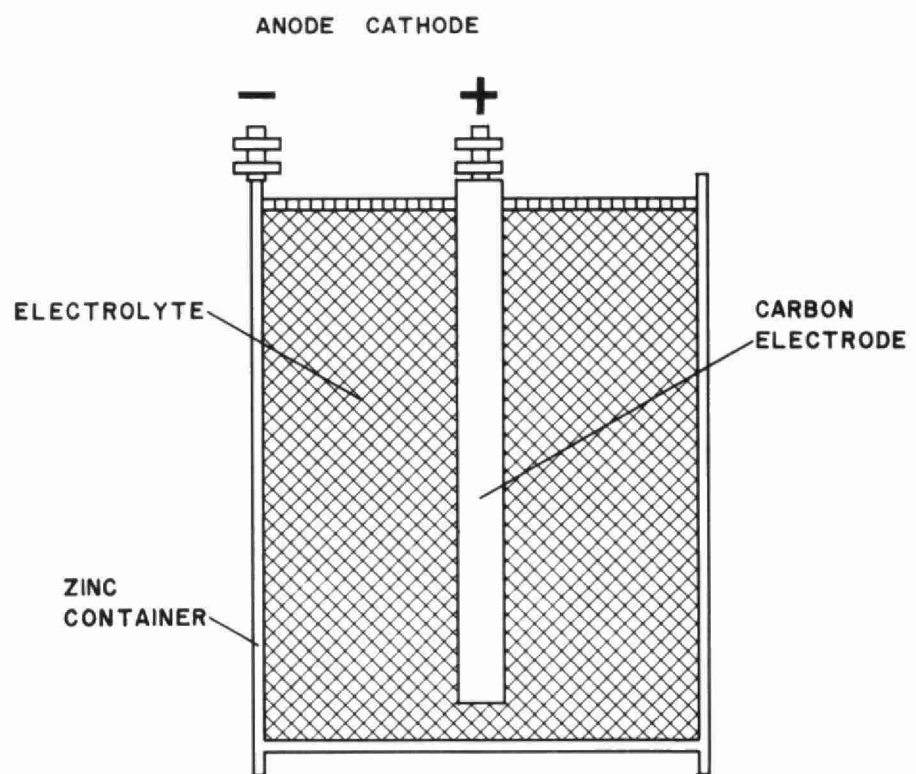
If a wire is connected between the two terminals, an electrical circuit is completed and an electrical current will begin to flow as a result of electrochemical reaction taking place in the dry cell. At the anode, the zinc metal becomes dissolved or eaten away by the chemical action of the electrolyte. This forms positively charged particles and negatively charged particles (electrons).

metal—————→positive ions + electrons

The positively charged particles detach themselves from the surface of the metal and enter into the solution as positive ions while a corresponding number of electrons are left behind in the metal. This surplus of electrons makes the anode the negative terminal and develops a potential difference between the two terminals. If a wire or metallic conductor is placed in between the terminals, there will be a movement of electrons causing a flow of an electrical current. In the meantime, the positive ions migrate through the solution towards the cathode where they become neutralized by the electrons and plate out on the surface of the carbon electrode.

The electrical current will continue to flow as long as electrons are produced by the dissolving action of metallic zinc at the anode.

FIGURE 1



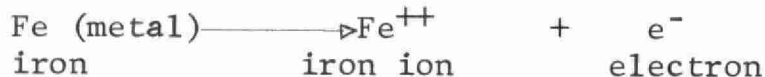
SCHEMATIC DIAGRAM OF A DRY CELL

Fundamentally, the corrosion of a metal is also an electrochemical process, similar to the reactions taking place in a dry cell and it is always accompanied by a flow of electric current. In order for corrosion to occur, there must be water or moisture in contact with the metal. There must be anodic (negative) areas on the metal where the electrons are released with the formation of positive metal ions and cathodic (positive) areas where the electrons are taken up simultaneously either by the neutralization of positive ions or formation of negative ions. In other words, action at the anode will not take place unless there is another action going on simultaneously at the cathode.

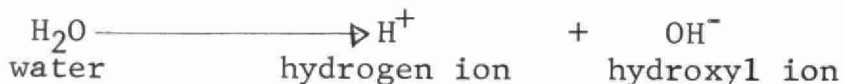
An example of metallic corrosion will be illustrated with the corrosion of iron.

CORROSION OF IRON IN WATER

When iron corrodes in water one area becomes an anode and another a cathode such as in a dry cell. This is known as a corrosion cell. At the anode, iron dissolves into water as positive Fe^{++} ions leaving behind electrons in the metal. (Fig. 2)



The electron travels through the metal to the cathode. Water is an electrolyte which dissociates into positive H^{+} ions and negative OH^{-} ions.

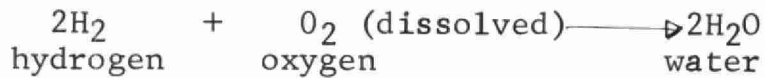


At the cathode, the H^{+} ions in the water react with the electrons to form H_2 gas. (Fig. 3)

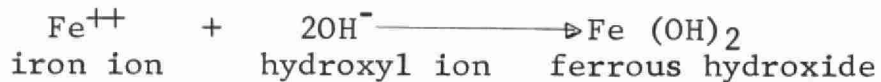


This action leads to the formation of hydrogen film on the metal surface at the cathode. This is called "polarization" (Fig. 4)

When the surface of the cathodic area becomes completely covered with H_2 gas or polarized, the electrochemical reaction stops unless the hydrogen film can be removed by scouring or oxidized by the dissolved oxygen.



As a result of H_2 gas forming at the cathode, there is a surplus of OH^- ions which tend to move towards the anode. During their migration through the electrolyte, they encounter Fe^{++} ions moving in the opposite direction. These ions combine to form ferrous hydroxide. (Fig. 5)



In order to maintain electrical neutrality in the solution, more Fe^{++} ions must be released to neutralize the excess OH^- ions remaining. This action causes more corrosion of the iron at the anode. The ferrous hydroxide reacts with the oxygen dissolved in the water and forms an insoluble precipitate which is the familiar iron rust or ferric hydroxide.

CONDITIONS AFFECTING CORROSION RATES

Corrosion will occur if the metal has an anode and a cathode in contact with each other in an electrolyte to allow the passage of an electrical current. The rate of corrosion is dependent upon the various conditions.

(a) Galvanic or Bimetallic Corrosion

When two dissimilar metals in contact with each other are immersed in a corrosive water, a galvanic cell may set up. It is likely that one of the metals may undergo corrosion more rapidly than the other. The rate of corrosion depends on, in part, the degree of electrical dissimilarity between the metals in the electromotive series as shown below.

Table of Electromotive Series

Magnesium	Corroded End
Zinc	(Anodic)
Aluminium	
Steel or Iron	
Cast Iron	
Nickel	
Tin	
Brass	
Copper	
Silver	
Gold	Protected End
Platinum	(Cathodic)

This table gives the relative position of various metals which will enter into bimetallic corrosion. The metal at the top of the list corrodes more quickly when placed in contact with any one below it. For example, iron and steel would corrode more quickly than either nickel or copper.

It should be noted that not all of the dissimilar metals placed in an electrolyte will undergo corrosion but the rate of corrosion is dependent on a wide range of conditions.

(b) Single Metal with Dissimilar Surface Characteristics

A single metal with some differences in physical or chemical characteristics on its surface may influence its rate of corrosion. For example, the corrosion can be caused by an electric current set up between two portions of the same metal of which one part is not under stress and the other is stressed. Other examples include a metal with its oxide or mill scale and cast iron in which the electrochemical reaction takes place between the iron acting the anode and the graphite as the cathode.

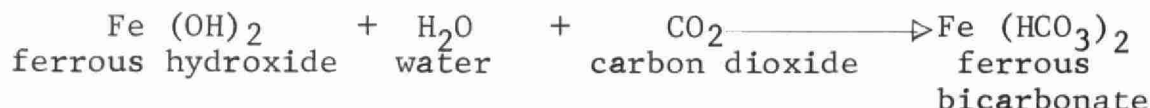
(c) Chemical Quality of Water

The rate of corrosion can be greatly influenced by the chemical quality of water. The presence of certain ions or molecules capable of reacting with the metal may enhance the corrosive nature of water.

Waters with low values of alkalinity, hardness (calcium content) and pH tend to be slightly corrosive to iron. The presence of dissolved gases such as oxygen, carbon dioxide and hydrogen sulphide in the water will substantially increase its corrosive properties when in contact with iron.

Dissolved oxygen plays an important role in corrosion because it depolarizes the cathode by removing the layer of hydrogen gas that is formed there. This permits the continuation of further corrosion. However, oxygen, depending upon the presence of carbon dioxide, may also react with ferrous hydroxide to form a protective coating of ferric hydroxide to protect the metal against further attack.

Carbon dioxide will react with ferrous hydroxide and form a soluble ferrous bicarbonate.



The product thus formed, dissolves and exposes the metal surface to further attack.

Carbon dioxide produces carbonic acid with water which tend to reduce the pH and increase the corrosiveness of the water.

Hydrogen sulphide will attack iron directly or it may form sulphuric or sulphurous acids which, in turn, will corrode the metal.

(d) Micro-organisms

There are several species of micro-organisms which are responsible for corrosion problems particularly those involving ferrous metals. These include sulphate-reducing bacteria and iron bacteria. Sulphate-reducing bacteria, under anaerobic conditions (absence of oxygen), react with various sulphates and sulphur-containing organic substances to produce hydrogen sulphide. These bacteria do not cause any corrosion directly but the hydrogen sulphide they produce, will attack the iron in watermain or it may combine with the water to form sulphuric acid that will react with the metal. Iron bacteria consume the iron and iron compounds dissolved in water and lead to conditions that will accelerate corrosion.

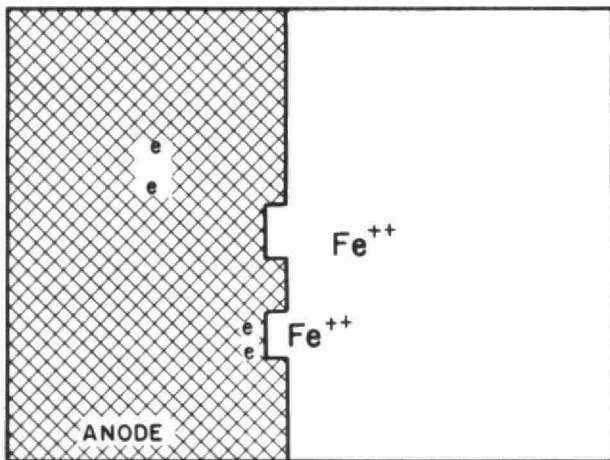


FIG. 2

FORMATION OF Fe^{++} IONS THE ANODE IN
CORROSION OF IRON

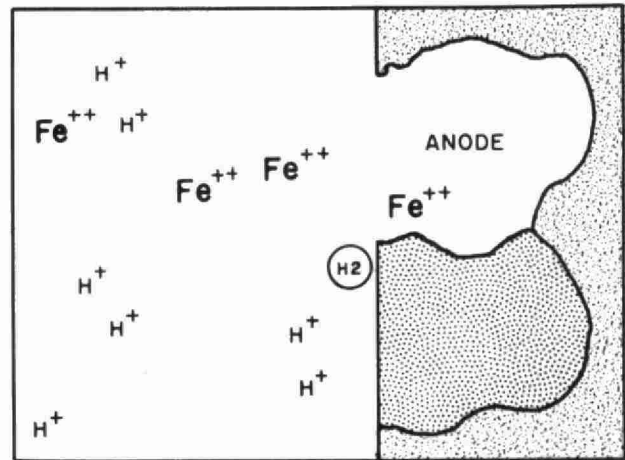


FIG. 3

FORMATION OF H_2 GAS AT THE CATHODE

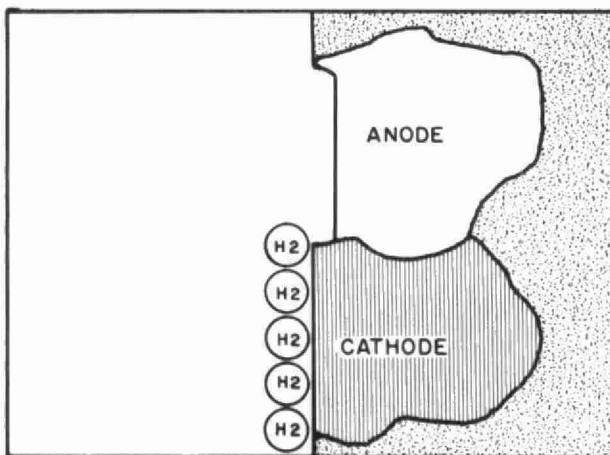


FIG. 4

POLARIZATION OF CATHODE BY A FILM OF
 H_2 GAS

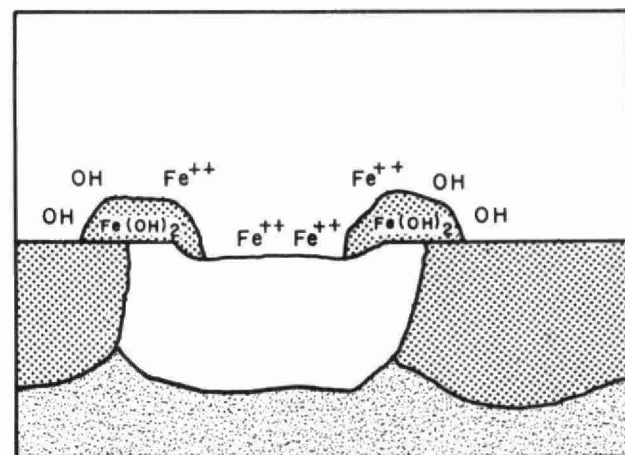


FIG. 5

FORMATION OF FERROUS HYDROXIDE IN
THE RUSTING OF IRON

Bacteria are also responsible for the decomposition of organic compounds present in the water. This results in the production of organic acids which, in turn, may cause localized corrosion in the distribution system.

(e) Flow Velocity of Water in Pipes

At high velocities, the water has a tendency to prevent the deposition of a protective coating on the metal surface and continue to expose the metal surface to further corrosion.

(f) Temperatures

In waters at higher temperatures, the metals generally corrode more rapidly.

SIGNS OF CORROSION IN DISTRIBUTION SYSTEMS

Serious corrosion problems in distribution systems and watermains are indicated by the following observations:

- (a) discoloration of water resulting from the presence of rust or "red" water
- (b) formation of tubercles on the inner walls of pipes and watermains
- (c) loss of flow capacity in the pipes due to the accumulation of corrosion products deposited on the walls
- (d) presence of strong hydrogen sulphide odours particularly in the "dead-ends"
- (e) sudden decrease in dissolved oxygen content of the water.

METHODS FOR CORROSION CONTROL

Practical methods for controlling corrosion in water supply systems may involve any one or a combination of the following:

- (a) insulation of the anode from the cathode
- (b) protective coatings
- (c) removal of dissolved carbon dioxide
- (d) destruction of micro-organisms
- (e) chemical conditioning
- (f) cathodic protection

(a) Insulation of the Anode from the Cathode

(i) Avoid connecting a new steel or iron pipe directly to a rusty old pipe. New pipe becomes the anode to the old. Use an insulating flange at the junction between the old and new pipes.

(ii) Install insulating flanges between the joints if the pipe traverses a moist ground area adjacent to a dry area.

(iii) Use insulating fittings to ensure an electrical separation between any copper service lines and cast-iron, iron or steel watermain.

(b) Protective Coatings

Corrosion in water supply systems can be reduced effectively by applying a protective coating on the surface of water pipes in order to prevent any direct contact between the metal surface and the water.

Protective coatings can be applied physically or chemically on the surface of the pipe prior to installation or by the special treatment of water. For the protection of iron pipes, the metal surface is coated with a layer of another metal or one of several non-metallic materials. The metal selected for this purpose consists of zinc or some other less noble metal that will undergo corrosion more readily and permit the iron to become cathodic, thus protecting it against corrosion. The metal coating can be applied by metal plating techniques, spraying molten metal on the surface or in a hot dip process similar to that employed plating zinc on galvanized iron.

Non-metallic substances applied carefully on the surface of metal pipes also provide good protection against the corrosive actions of water and the surrounding soils. These include tar compounds, bitumastic enamels, special paints and some of the newer materials such as epoxy resins and plastic compounds.

In municipalities where aggressive waters are encountered, corrosion problems can be minimized by the use of asbestos cement (Transite) pipes or cast-iron pipes with cement linings.

It should be noted that despite the use of protective coatings on pipes, the water will remain corrosive in character and may corrode other unprotected components of the distribution system such as the plumbing fixtures.

Protective coatings on metal pipes can also be applied by chemical means. This involves the deposition of a protective film by continuous addition of certain chemicals to the water. These include sodium silicate, sodium hexametaphosphate (Calgon) and calcium carbonate. The use of these chemicals offer some advantages because they will provide protection to the entire system at very low cost. However, these chemicals must be added continuously and their effectiveness in controlling corrosion has not been firmly established.

Sodium silicate has been found to be effective as a corrosion inhibitor in the presence of calcium and magnesium salts. It has been used in the treatment of industrial and hot water supplies but rarely in public water supplies.

Sodium hexametaphosphate or glassy phosphate has been employed extensively in the treatment of corrosive waters. Its application in concentrations of 1 to 2 ppm has been found to be very effective in corrosion prevention for some water supplies. Conflicting results have been reported with regards to the effectiveness of this method of treatment. However, it is well-established that with the use of this chemical, the precipitation of any iron, already present or that due to corrosion can be prevented, the formation of large tubercles on the surface of the pipe can be avoided and hence the pipes will not become clogged with the products of corrosion. Addition of sodium hexametaphosphate in excess may be detrimental rather than helpful. Not all water supplies can be successfully treated by this method. Therefore, it is advisable to conduct some experiments to determine its usefulness for each application.

Calcium carbonate, if applied under favourable conditions of pH and alkalinity in the water will deposit a good protective film on the surface of the pipe.

(c) Removal of Dissolved Carbon Dioxide

High concentrations of carbon dioxide can be best removed by aeration to about 5 ppm. However, aeration may not prevent corrosion because the oxygen introduced will likely counteract any benefits derived from the reduction of carbon dioxide.

Corrosive effects of waters containing low concentrations of carbon dioxide can be reduced by the use of calcium hydroxide or some alkali chemical which will increase the pH.

(d) Control of Microbial Activity

In order to control the growths of corrosion-causing organisms, good chlorination practices should be followed. New watermains should be thoroughly disinfected with high dosages of chlorine before placing into service. Troublesome waters should be treated with an adequate dosage of disinfectant to maintain a proper level of chlorine residual in the distribution system.

(e) Chemical Conditioning

Chemical conditioning means the use of chemical additives to inhibit or reduce the corrosive effects of water. One of the most effective methods is to raise the pH value of the water by treatment with alkali chemicals. Caustic soda, lime, soda ash or beds of crushed limestone are commonly used but the choice of the chemical is dictated by such factors as cost, hardness (calcium content) and carbon dioxide content of the water and its ultimate use.

Caustic soda is the most efficient for raising the pH of the water but it is relatively expensive and it does not offer any protective coating for the metal as the lime does. Lime and crushed limestone are cheap chemicals that can improve the pH as well as provide a protective coating under proper conditions but it may also increase the hardness of water. Soda ash is reasonable in cost and efficiency and it will not cause any hardness but it does not form any protective coating.

Some chemicals are added to the water as corrosion inhibitors which help to remove dissolved oxygen and carbon dioxide. Most of these are utilized in the treatment of corrosive waters in hot water systems of buildings and are rarely used in treating public water supplies.

(f) Cathodic Protection

Cathodic protection involves the use of an applied electrical current to protect the metal surface against corrosion. The pipe or metal structure to be protected is made cathodic by connecting it in an electrical circuit to the negative pole of a DC generator. The positive pole is connected to anodes buried in the ground as shown in Figure 6.

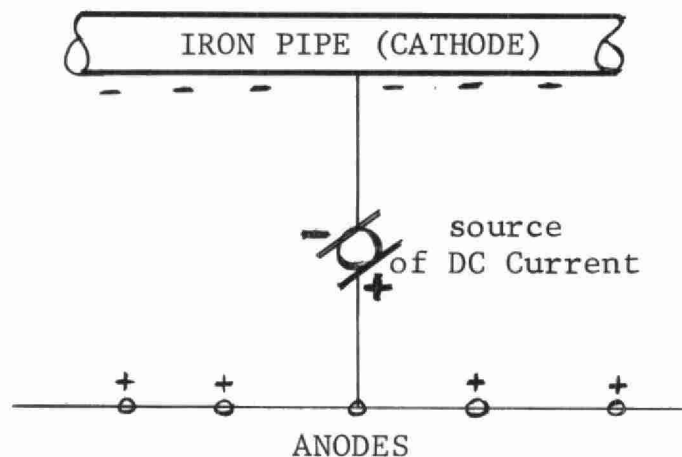


Figure 6

Electrical Circuit for Cathodic Protection

Cathodic protection is an excellent method for protecting steel water tanks and the outside walls of pipes buried in the ground. However, it does not always prevent corrosion in the insides of the pipes.

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REVIEW QUESTIONS

- (1) Under what conditions will iron corrode?
- (2) What makes water more corrosive?
- (3) List some of the problems caused by corrosion in a waterworks system.
- (4) What steps should a waterworks operator take at the plant to reduce corrosion problems?

CHEMICAL ANALYSIS OF WATER

Iron, Hardness, and Chlorine Residual

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INTRODUCTION

When a source of drinking water is examined by chemical analysis we are not so much interested in the factors which indicate organic pollution as we are in those factors which indicate reasons for colour, taste, or odour problems. In general these factors are not dangerous to human life, but they can be a source of irritation to the consumer. One of the purposes of water treatment is the removal of these nuisance parameters wherever possible.

Before treatment can be considered however, we must know how much of the particular material is present. Since many of the treatment procedures involve the addition of other chemicals, we must at all times avoid adding an excess of the cure for fear of creating new problems which may be even more difficult to correct.

Chemical analysis permits us to measure both the level of the nuisance factor, and after treatment, the level of chemical additive remaining. Several analytical techniques are used in determining the amount of a material present.

Iron is determined by colorimetric analysis. An excess of reagent which forms a coloured compound in the presence of iron is added to the sample and to a series of standard solutions containing known amounts of iron. The amount of iron in the sample is determined by comparing colours.

Hardness is determined by titration of the sample in the presence of an indicator. The titrating reagent is added to the sample until the indicator just changes colour. If we know the strength (or concentration) of the "titrant" and the volume required, we can calculate the amount of Hardness.

Chlorine Residual is also determined by colorimetric analysis, but in this case the standards for comparison are in the form of permanent, standard, coloured disks rather than standard solutions. In this test use is made of the fact that rate of reaction (i.e. rate of colour formation) depends on a number of factors.

IRON ANALYSIS

Everyone is familiar with the metal iron. When iron is exposed to the oxygen of the air it "rusts", or in chemical terms it is "oxidized" to form iron oxides. The iron in these oxides may be in one of two states, the less oxidized ferrous ion (Fe^{++}) or the more oxidized ferric ion (Fe^{+++}). The chemical symbol for the metal iron is Fe. The number of plus signs indicates the number of negatively charged electrons which have been removed during oxidation to form the ion.

In water, iron is present in both of the above ion forms. The main point to remember is that dissolved oxygen in the water rapidly converts the soluble ferrous iron into the less soluble ferric iron which then settles out of solution creating stains in sinks and on clothes. Because of these rapid changes in the ratio of ferrous to ferric iron, we are normally only interested in the total amount of iron in the sample.

The phenanthroline method of analysis for iron depends on the formation of a coloured complex between the analytical reagents and ferrous iron. The procedure for Total Iron is described on pages 156 to 159 in "Standard Methods" - 12th edition. It is necessary to:

- a) Bring all the suspended iron into solution by boiling in acid for the specified time.
- b) Convert all the ferric iron into the ferrous by "reducing" it (the reverse of "oxidizing") with hydroxylamine reagent.
- c) Analyse the ferrous iron so formed immediately with phenanthroline reagent which will form an amount of colour proportional to the amount of ferrous iron present.

In some cases only the amount of dissolved iron is desired. This can be only obtained by filtering out any suspended material in the fresh sample and analyzing. Of course, if the sample is allowed to stand before filtering, some of the soluble (dissolved) iron settles out due to oxidation and will be removed in the filtration step.

If only the ferrous iron content of the sample is desired, the "reducing" step using hydroxylamine is left out. Standard Methods recommends taking samples for this analysis in bottles to which the colour developing reagents have been already added.

Discussion of Reagent

- Standards: With proper care these are fairly stable and freshly prepared standards need not be prepared with each group of samples. However an individual standard should be tested frequently to ensure continued accuracy.
- Blanks: When any new reagent is put into service, a blank analysis should be made on iron-free water to determine how much iron is being contributed by the reagents which will always be contaminated with varying traces of iron.
- Hydroxylamine: This is the reducing agent used to convert ferric iron to ferrous iron. When this reagent is omitted, only ferrous iron in the sample is measured.
- Phenanthroline: This is the colour-producing reagent which forms an orange-red complex with ferrous iron, between pH 2.9 and pH 3.5. The pH is adjusted to this range by the use of buffers as described in Standard Methods.

WATER HARDNESS

Hardness of water is a measure of the capacity of water to form curds or scum (the ring-around-the-bath tub) in the presence of soap. The most common cause of this phenomenon is the presence of calcium and/or magnesium ions although other metal ions such as iron, aluminum and zinc can also form precipitates with soap.

Since the most common source of calcium and magnesium in water is due to erosion of limestone (carbonate) rock by rain, Total Hardness is measured in terms of calcium carbonate (CaCO_3). This does not necessarily imply that all the hardness is due to calcium or that carbonate is present in the water.

Rather than use soap to "titrate" (measure) Hardness the more convenient and accurate E.D.T.A. method is used. In this method a solution of E.D.T.A. (ethylene-diamine-tetra-acetic acid) is added dropwise to the sample until all calcium and magnesium ions have been "tied up" in the form of a stable, soluble, nonreactive "chelate" compound.

If we know the volume of E.D.T.A. added, and the concentration of the E.D.T.A., i.e. amount per unit volume, we can calculate the amount of calcium and magnesium in the sample. But how do we know when all the ions have been "tied up"? When have we reached the end of the titration?

Eriochrome Black T is a blue coloured dye which can be used as an "indicator". When a few crystals of this dry powdered chemical are added to a sample at a pH of 10 it reacts with a few of the magnesium ions to form a wine red colour. As the E.D.T.A. titration is performed in this red solution, first the free calcium ions, and then the free magnesium ions are "tied up". Finally when all of these free ions have been titrated, the E.D.T.A. reacts with the magnesium ions which had reacted with the Eriochrome Black T. The indicator thus changes from the wine red colour, back into its blue form thereby signalling the end of the titration.

Discussion of Reagents

Standard E.D.T.A.: Prepared and standardized by titration against a calcium carbonate solution of exactly known strength. Usually 1.00 ml. of E.D.T.A. solution reacts with exactly 1.00 mg of CaCO_3 .

Eriochrome Black T: 0.5 gms of the dye are mixed thoroughly with 100 gms of finely ground reagent grade NaCl (sodium chloride).

Buffer: A variety of pH buffers have been developed. Many of these contain ammonia and gradually lose their strength as the ammonia evaporates. The main supply must therefore be kept tightly stoppered and a portion transferred to a small bottle for use.

Calculation: Say 2.9 mls of E.D.T.A. were required for 100 mls of sample. This is equivalent to 2.9 mg CaCO_3 since 1.0 ml E.D.T.A. = 1.0 mg CaCO_3 .

$$\begin{aligned} \therefore \text{Hardness} &= 2.9 \times \frac{1000}{100} = 29. \text{ mg/l} \\ &= 29. \text{ ppm} \end{aligned}$$

If 2.9 mls of E.D.T.A. is required for 20 mls sample

$$\begin{aligned} \text{Hardness} &= 2.9 \times \frac{1000}{20} = 145. \text{ mg/l} \\ &= 145. \text{ ppm} \end{aligned}$$

CHLORINE RESIDUAL

It is necessary to know how much chlorine (Cl_2) is left in a treated water after it has been given time to react with the nuisance factors we want removed. Usually we are interested in the residual after 15 minutes contact time either in the plant or in the sample bottle. The residual chlorine may be present as free available chlorine or as combined available chlorine. In addition there may also be present certain materials which will interfere with the analysis by giving a false colour. It should be noted that chlorine in solution is very unstable. Analysis should be started at the specified time and excessive agitation of the sample or exposure of the sample to direct sunlight should be avoided.

For a thorough discussion of the factors involved in determining Chlorine Residual it is strongly suggested that a copy of Standard Methods - 12th edition be obtained and studied. (pages 90-91 and 93-100).

Note: Orthotolidine reagent is unstable. Do not use rubber stoppers on the stock reagent bottles. Store the solution in a dark cool place and discard after 6 months or if it turns yellow.

When analyzing - always add sample to reagent to avoid local pH greater than 1.3 in the solution during mixing.

Avoid direct sunlight at all times during analysis. Use "north lighting" or artificial lighting as described in Standard Methods.

Total Available Chlorine

Warm sample to 20°C and add sample to the reagent. Read colour after 3 minutes or at maximum.

At room temperature (20°C) the addition of sample to orthotolidine reagent produces a colour development due to the free available chlorine which reacts instantly and also the combined available chlorine which reacts more slowly. At this temperature, maximum colour development is reached in about 3 minutes and then the colour starts to fade.

Remarks

It should be noted that not all of the colour developed is due necessarily to available chlorine. Other materials such as algae, ferric iron, nitrite will produce an interfering or false colour. Therefore a sample of dechlorinated water should also be tested to determine the amount of interference, i.e. "false" chlorine, present in the sample.

PUMPING OF WATER

A. B. Patterson, Director

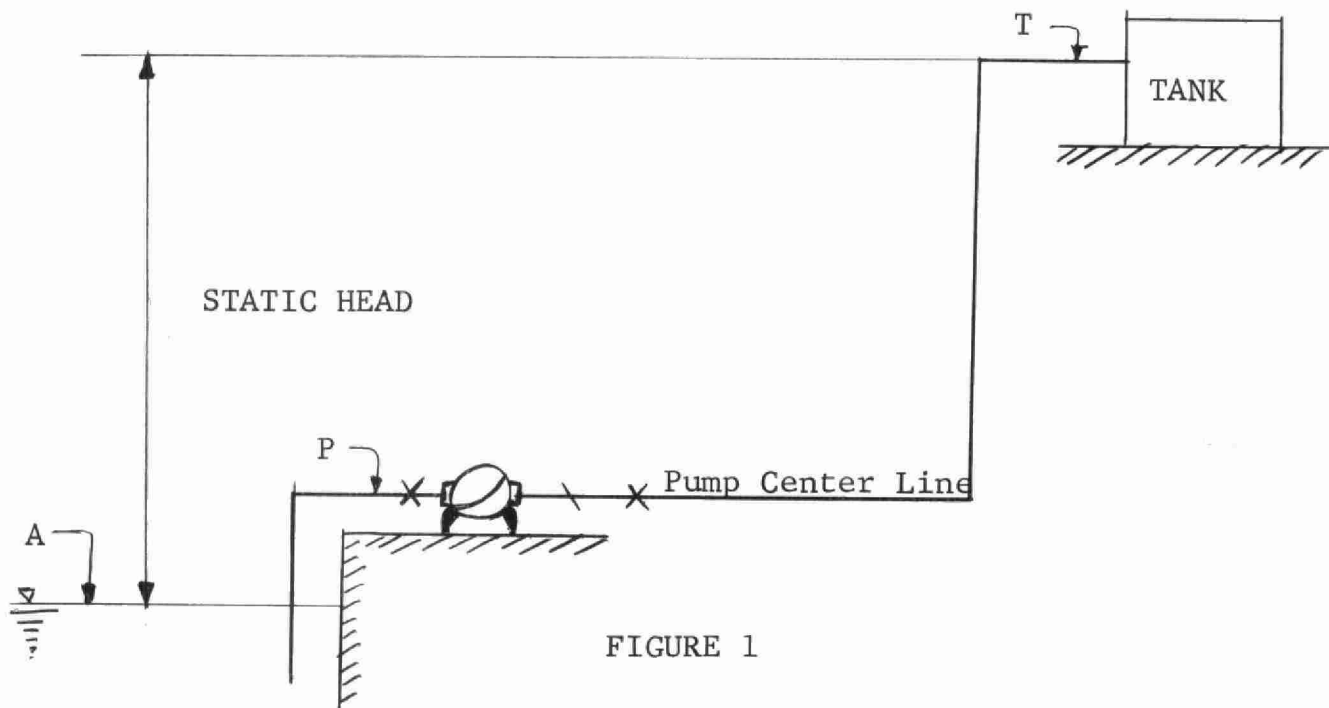
James F. MacLaren Limited

SYSTEM HEAD CURVE

One of the most important concepts to be understood in the correct application of centrifugal pumps is that of "system head", the natural opposition to flow inherent in any piping-process installation. In feeding into a network of piping, a pump works against a head imposed upon it by that network. The overall network is called, for our purposes, the "system" and it includes the pipe, fittings, valves, tanks and other components which may be in the flow path. The "system head" thus includes the various static (vertical) heights through which the fluid must be lifted in passing through the system as well as the friction losses occurring in the components as a result of that flow.

To illustrate, let us imagine a condition where we want to bring water at a certain rate from a source of lower elevation to some reservoir or water tank at a distant site and at a relatively much higher elevation. To do this, in the most practical and economical way, is to use a pump to move the water from the source thru the pipe and eventually into the reservoir or tank. But before we could say how big or what type of pump should be used, we have to find out how much energy is to be imparted to the water in moving it to the elevated tank; or plainly, we might ask what are the obstacles or elements we have to work against in moving the water from one point to another.

In Figure 1, we have that above mentioned condition. Here, we are required to pump water from source A to an elevated tank. The vertical distance, which the water has to travel, measured from the free top water level at A to the highest point of free discharge at tank is called the "static head". In this particular case, it is represented by distance $\overline{AT}$. Here it should be noted that static head is made up of the static lift, which is represented by distance $\overline{AP}$ and static discharge head by $\overline{PT}$.



Another element to overcome is friction, which all of us know is a kind of force which tends to slow down a body in motion. In our system, that force takes the form of a resistance to flow of water in pipes and fittings. We shall call that force or pressure required to overcome such resistance as the "friction loss" or "friction head", and its magnitude in terms of feet of the liquid being moved varies as the amount or rate of flow of the liquid.

The combined effects of the vertical distance and friction against the effort or pressure in moving the water is called the system head. Expressed arithmetically, it is the sum of the static head, and the friction loss at a given flow represents the system head at that capacity.

We shall now try to present these conditions graphically to appreciate at a glance what a system head is, and, at the same time, know the characteristics of that system.

If we try to pump even an infinitesimally small amount of water from the source to the tank, we still have that vertical distance or static head to overcome, assuming here that the friction loss is negligible or almost equal to zero. As we increase the amount of water or rate of flow, we find out that the friction loss will also start to increase, hence the magnitude of the system head.

These values, when plotted with head as the ordinate and capacity as the abscissa, result in a curve called the "head curve", as shown by Figure 2. It is also interchangeably referred to as the "system demand curve".

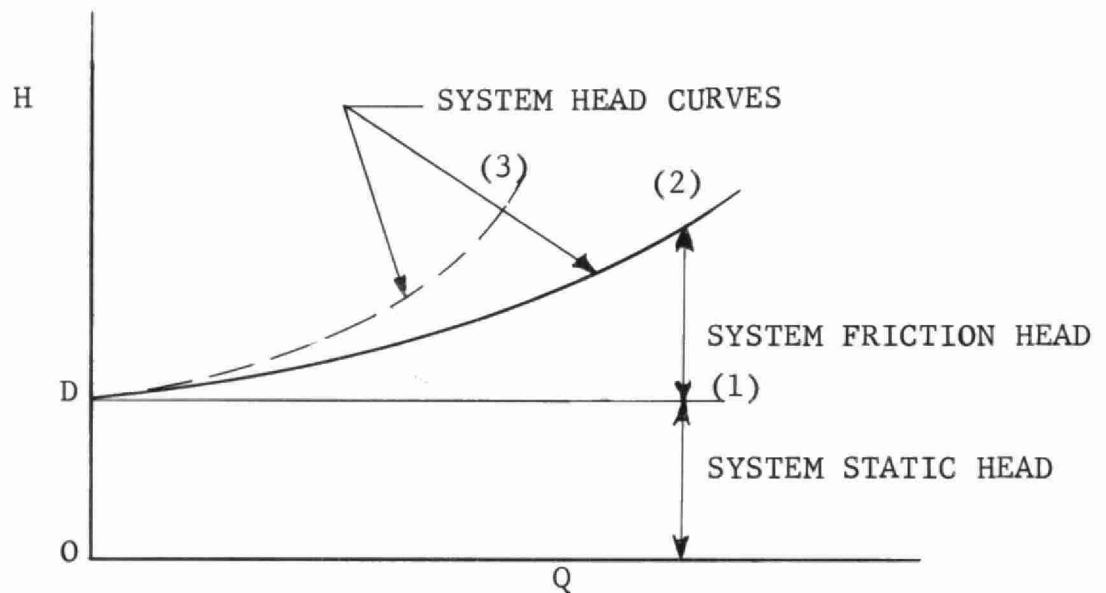


FIGURE 2

If there were no friction losses along the piping, the system head curve would be represented by a horizontal straight line shown by line (1). With friction losses, the curve will be line (2). Now, if we try to use another piping system, this time maybe with smaller pipes and fitting, the friction loss will increase for the same amount of flow and that is shown by the dotted line (3).

Note that the system head curve has a parabolic form as would be expected, since friction loss varies roughly as the square of the change in capacity or, assuming a constant cross-sectional area of pipe, friction loss varies roughly as the square of the change in liquid velocity through the pipe. at the outset, it must be understood that any value assigned to friction loss can only be an approximation since friction losses will vary with size and type of pipe and fittings, with the interior condition of the pipe and with the character of the fluid being handled.

It will be noted that if there were no static head involved in the system, and there is only the friction head to work against, then point D in Figure 2 would shift down to zero, as shown in Figure 3.

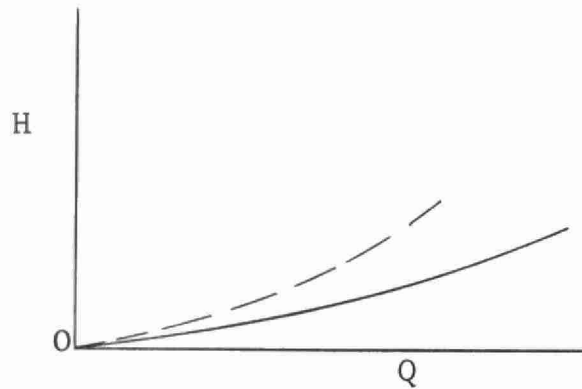


FIGURE 3

USING THE SYSTEM HEAD CURVE

Knowing the total head that we have to work against to move the required amount of liquid, we can now proceed in selecting the pumping unit. At this point, you should all be familiar with the basic fundamentals involved in pump operation, as discussed in the basic course.

Assume now that we have considered a particular pumping unit with a known head-capacity characteristic curve, and try to match it with the required flow Q_A and head H_A on the system head curve. This can be easily done by superimposing both curves, as in Figure 4.

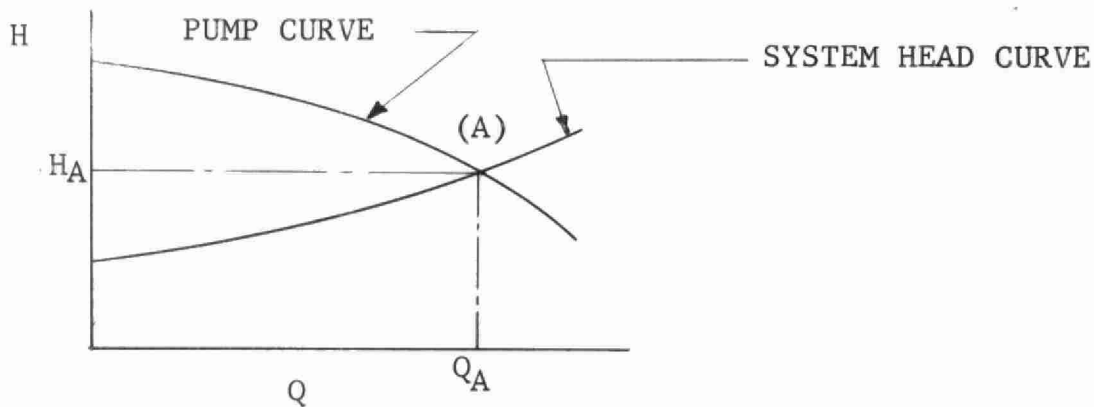


FIGURE 4

If the pump is rated at the capacity and head where the pump curve intersects the system head curve, then we can say that the pump matches the system.

You should always remember that the operating conditions for a pump can best be determined by developing an actual graph by plotting the system head curve and the pump characteristic curve to the same scale. The operating point will always be at the intersection of the two curves. All pump installations can be represented graphically as being a system head curve upon which the characteristic head-capacity curve of one or more pumps has been superimposed. Some systems are more complicated to calculate than others; these may require the calculation of individual portions of the system separately for later addition graphically to obtain the overall system head curve.

PARALLEL OPERATION

Since examples will lend further clarification, let us consider several that are typical of common applications. Often, an installation will include problems of capacity variation over a wide range. This can be accomplished by placing pumps in parallel to discharge at the same pressure into the same header. Each pump can be placed in service or removed according to the system demand at the time. One approach would be the use of identical pumps placed in parallel. These would each develop the same head at any given capacity. When operating in parallel, the total capacity of two such pumps would be double the capacity of one pump at the head involved. This is illustrated graphically by Figure 5.

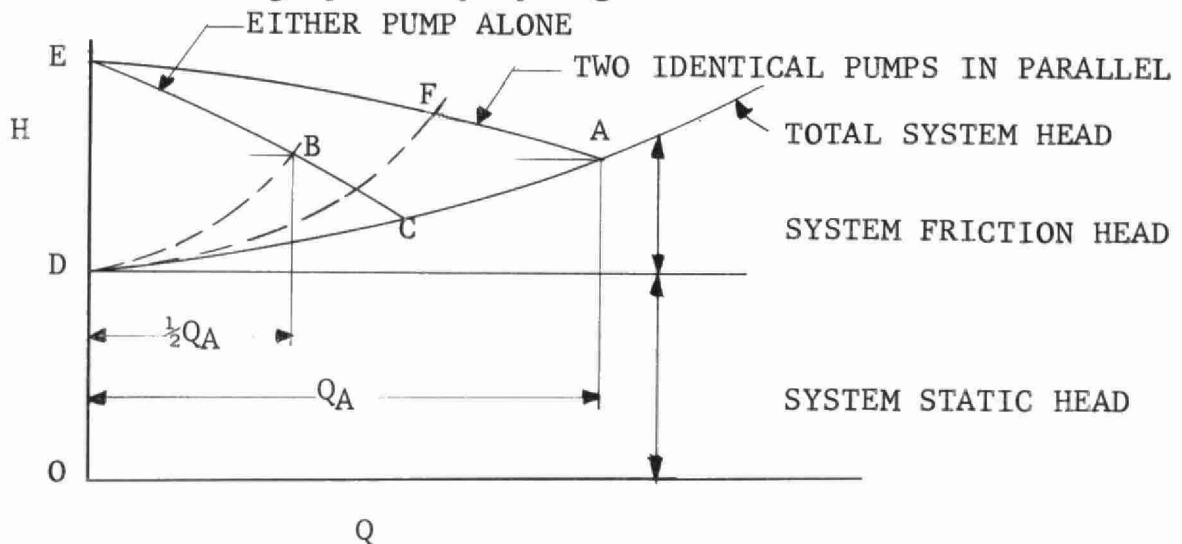


FIGURE 5

The two pumps together operate on curve EA intersecting the system head curve at point A. Note the combined operating curve starts at the no-flow or "shut-off" condition, E, of a single pump. Each succeeding point on the curve is developed by doubling the capacity of a single pump at any given head.

Were a single pump to operate alone at its design capacity, and, therefore, at a head of A, its discharge valve would require throttling to create sufficient artificial friction loss to form a system curve DB. If the demand were to increase, the throttle must be made less severe until point C is reached. The system itself would limit the flow to this latter capacity. The determination of point C is important for, with it, all possible operating points of a single pump have then been determined and the pump's driver can be sized to handle the maximum power requirement within the range EC. Even if the pump has been factory tested and has demonstrated far greater carry-out abilities and, depending upon the design, possibly greater power requirements, point C of the system head curve has determined the maximum power required for the pump when placed in the particular installation under consideration.

The development of the system head curve and a knowledge of demand variations to be expected can be of great assistance in determining the most economical division of the entire range into pump sizes needed. This can be done ideally when laying out a new system. For paralleling of the pumps to be accomplished properly, the units should be selected to have the same or nearly the same shut-off head. Otherwise, one pump having the higher shut-off head would tend to back the others off the line.

Such a system is represented by Figure 6. Pump #1 has the lower shut-off head and would operate at the shut-off condition at any capacity demand less than that of point B. The combined curve of the two pumps in parallel would be FBG. Were the capacity to decrease to B or below, pump #1 operates at the shut-off condition. It could thus endanger itself if not shut down promptly, or if provision had not been made to by-pass adequate flow to carry away the heat that would be generated.

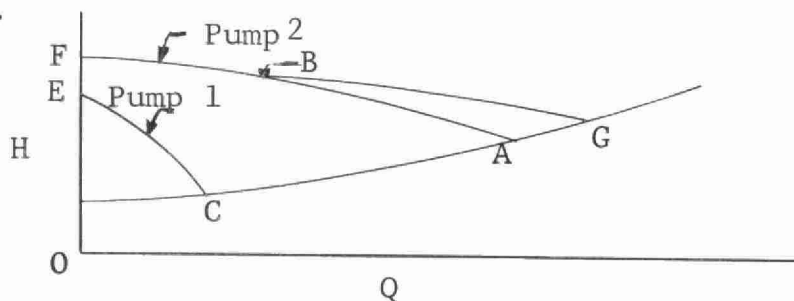


FIGURE 6

This arrangement is somewhat typical of municipal water works systems that have experienced an expansion of facilities due to population growth in outlying areas necessitating longer lines, often extensions of older mains now too small but too expensive to replace. The result is increased friction requiring additional pumps of larger capacity and higher head as well to fulfill the increased demand.

It should be noted further that, in this example, the larger pump would be required to operate over the range of low capacities where it would not be at its best efficiency point, nor possibly as efficient as a smaller pump designed for those capacities. A solution to this aspect of the problem, as well as the matter of paralleling properly, is often found in the purchase of a new impeller for the smaller pump, this impeller having a steeper head-capacity characteristic, or, in other words, a higher shut-off head. Another possible solution may lie in stepping up the speed of the small unit, since head increases by the square of the change in speed of the pump. While a new and larger motor may be required, the cost will be easily justified in most cases, and will be considerably less than the purchase of a complete new pumping unit. In contemplating such a modification, full investigation of the particular pump's characteristics and design must be made with the manufacturer.

CAVITATION

Cavitation can be a serious problem in pump performance and should be understood by all persons having supervisory positions.

For a relatively simple practical demonstration we may turn to the Venturi tube as a basis of understanding. When a liquid such as water is forced past a local restriction such as a Venturi meter throat, a region of lowered pressure is formed in the narrow part of the tube or throat. As the water cannot expand or support tension, stress and, hence, cavities form. Actually, boiling has been achieved at the greatly reduced pressure. The boiling bubbles contain vapour and some very low pressure air.

Theoretically, spherical bubbles collapsing in a fluid would deliver a peak pressure that is actually infinite. Nothing on earth is perfect, but we do know that bubble collapse is accompanied by billions of hydraulic blows and great force. On the boundary or surface areas a shock is radiated which not only erodes the surface, the tremendous force released is also able to cause eventual fracture of brittle materials and fatigue failure of ductile materials.

SHAFT STRESSES

1. Radial Thrust

Due to the shape of a centrifugal volute casing, if a pump operates at other than its design capacity, a certain unbalance of the hydraulic forces acting radially on the impeller takes place. The maximum unbalance occurs generally at zero capacity and is reduced as rated capacity is approached. This unbalance creates a radial load on the pump shaft. Some cases of shaft failure have been clearly traced to this cause.

2. Radial Stresses

Axial hydraulic thrust is the summation of unbalanced forces on an impeller acting in the axial, or "along the shaft" direction.

LEGAL PROBLEMS IN THE SUPPLY
AND TREATMENT OF WATER

H. Landis, Senior Solicitor

Legal Branch - OWRC

Under sections 2, 4 and 5 of The Public Utilities Act R. S. O. 1960, C. 335, municipalities are given broad power to establish, construct and operate waterworks such as treatment plants, storage reservoirs, wells, pumping stations and watermains, for both transmission of water from the source of supply to the municipality and for distribution in it. This authority includes the power to acquire or expropriate such land, waters, water privileges and rights of diversion as may be necessary for the purposes of the waterworks. Since the recent amendment (Statutes of Ontario 1962-63, C. 120, S. 1) which repealed the previous limit of 15 miles of the municipality, these powers may be legally exercised anywhere in Ontario.

Since these powers are permissive a municipality is not liable for an act of non-feasance in the operation of its waterworks. Thus in Seguin v. The Town of Hawkesbury (1955) O.R. 956 the municipality was held not liable for its failure to stop a fire due to the water pressure in the mains being too low because it allowed the fire alarm signal in the pumping station to fall into disrepair. Alternatively the Court held that the municipality's exercise of its power to operate a waterworks was not the cause of the loss, i.e. the cause was the fire and not the failure to supply water at an adequate pressure in the mains. The Court implied that if a municipality negligently exercised its power to establish and operate waterworks and such negligence caused fresh injury or loss which would not have occurred if the power had not been exercised the result would have been different. For example, a municipality might be liable if it negligently allowed the pressure in its watermains to rise to the point where damage to private plumbing resulted.

Under The Municipal Act R.S.O. 1960, C.249, sections 24, 377(5) & (6), 379(1) (paras. 52 & 82) provide certain powers regarding the joint operation or the extension or improvement of waterworks or for obtaining investigations or engineering reports in regard to waterworks or water supply systems. Sections 381(15) and 391(6) contain powers in regard to the regulation of water tanks, towers and mains. These sections under The Municipal Act do not confer any powers on a public utilities commission.

Under s. 38(1) of The Public Utilities Act a municipality may transfer the control and management of its waterworks or the construction of such works about to be established to a commission to be called "the public utilities commission of the (name of municipality)". The commission is a body corporate separate from the municipality. (s.38(1) and s. 42(1)).

Under section 41(1) of The Public Utilities Act (hereinafter referred to as the "Act") subject to the authority of the council with reference to providing the money required for the waterworks and the rights and powers of the council under The Local Improvement Act, all the powers, rights, authorities and privileges that are conferred by the Act on a municipal corporation must be exercised by the commission alone and cannot be exercised by the council (s. 41(1)). There is a question whether the powers of a commission under the Act are limited to those of "construction" and "control and management" of a waterworks or whether, subject to specific limitation in the Act, the commission's authority covers all the powers of the municipality under the Act related to waterworks.

It is clear that for some purposes such as liability for negligence or on a contract, the commission exercises whatever powers it has under the Act on behalf of the municipality - it is the statutory agent and the municipal corporation is the principal; (Scott v. Hydro Electric Commission of Hamilton 7 O.W.N. 385 at p. 386; Young v. Gravenhurst 22 O.L.R. 291 at pp. 295 and 297, aff'd 24 O.L.R. 467). However an important difference between the common law relation of an agent to his principal and the relation of a commission to a municipality and its council is that under the Act the council cannot require a commission to exercise any of the commission's powers, rights, authorities and privileges, and where a commission lawfully exercises them and where under the Act such exercise does not require the consent of the council, the council cannot directly control their exercise by the commission (s. 41(1)). A measure of indirect control may be exercised by the council refusing to provide any money required by the commission (s. 41(4)) or by the council repealing the by-law establishing the commission or entrusting it with the control and management of the waterworks, thereby terminating its existence (s. 38(6)).

Since the powers of a municipal corporation are exercised by its council (The Municipal Act R.S.O. 1960, C.249 s.9), the lack of direct control by the council means that, within the scope of its powers and where under the Act the consent of the council is not required, a commission has the discretion to make final decisions in regard to waterworks that may be contrary to the wishes of a municipality expressed through its council. This discretion to make such decisions is illustrated by City of Belleville v. The Public Utilities Commission of

Belleville (1949) O.R. 468, where the municipality brought an action to require the commission to raise a specified sum out of waterworks revenue to meet principal and interest instalments due under municipal waterworks debentures, and if such revenues were insufficient, to require the commission to impose a special water rate to raise the instalments and to pay over the instalments to the municipal treasurer annually. The Court held that the commission was not under any obligation to fix the special water rate or to raise out of its revenue the annual debenture payments, the reason being that under s. 41(1) the commission and not the municipality has the exclusive power to fix the rates and charges for the supply of water.

Similarly it would appear that a council cannot require a commission to make extensions, improvements, enlargements or other alterations to or in the waterworks since the commission, subject to obtaining the consent of the council to the works that the commission proposes to undertake under the circumstances set out in s. 41(5), has under s. 41(1) the exclusive power to determine whether such works should be undertaken and the nature of the works to be undertaken.

It is clear that the ownership of the waterworks cannot be vested in the commission at any time. (Belleville v. Public Utilities Commission of Belleville (1943) O.R. 87; Young v. Gravenhurst supra). In accordance with the concept of a statutory agent that has been applied to activities of the commission, it has been held that the commission's possession or occupancy of the waterworks is, in law, the same as the possession and occupancy of the municipality and that the knowledge of or attributed to the commission with respect to the waterworks is, in law, attributed to and considered as the knowledge of the municipality. Thus in MacKinder v. City of London (1953) O.R. 52, aff'd. p.59, the plaintiff recovered damages from the City of London for injuries sustained by falling into a hidden excavation in a public park owned by the City but under the control and management of the commission. The Court held that the commission's occupancy of the park is, in law, the occupancy of the municipality and that the knowledge of the commission's employees of the hidden excavation should be attributed to the commission and regarded, in law, as the knowledge of the municipality. Thus it would seem that legal actions that are required to be brought by or against the owner or the person having possession of waterworks should be brought by or against the municipality and not by or against the commission.

Fire and other insurance requirements for a waterworks should be considered in the light of the legal relation of a commission to the municipality. For example, because the commission has occupancy of the works in fact but cannot be the

legal owner it is desirable in a fire insurance policy either to have both the municipal corporation and the commission insured or to otherwise deal specifically with the ownership of the municipality and the occupancy of the commission. In addition insurance against liability for damage caused by the negligent operation of the waterworks should be procured in the name of the municipality because in the event of such liability it and not the commission is liable. (MacKinder v. City of London supra; Young v. Gravenhurst supra; Scott v. Hydro Electric Commission of the City of Hamilton supra; Stevens-Willson v. City of Chatham (1933) O.R. 305, aff'd 1934 S.C.R. 353, but the majority of the Supreme Court expressly left this question open).

For the purpose of liability for negligence the cases just cited held that the commission is a statutory agent of the municipality and that any action for damages based on the negligence of the commission itself must be brought against the municipality and not against the commission. An action for negligence may also be based on the acts or omissions of an employee of the commission. Generally speaking such an employee is, in law, a servant of the commission and not of the municipality. Since the Commission is a statutory agent, the municipality, as the principal, would be liable in accordance with the law of agency for the commission's servants' negligent acts or omissions only if the commission, as the agent, is responsible in accordance with the law of master and servant for such negligent acts or omissions. (see the dissenting judgment of Mr. Justice Crockett in Stevens-Willson v. City of Chatham (1934) S.C.R. at p. 370). In all cases of negligence the employee of a commission would be personally liable for the negligent performance of his duties.

For the purpose of liability on a contract MacDougall v. Windsor Water Commissioners (1900) 27 O.R. 566 aff'd. 31 S.C.R. 326 held that a municipality will be liable on a contract made by a commission as its statutory agent and acting within its authority. The Court implied that the action for breach of contract should be against the municipality alone and not against the commission. Although this case was not decided under the Act it is suggested that the liability of a municipality on a contract made by a public utilities commission under the Act would be determined in the same way.

Where the Act expressly requires the consent of the council to work by the commission (s.41(5)(b)) the authority of the commission to make the contract will require the consent of the council expressed by a by-law. Since the requirement of obtaining the consent of the council is unqualified it is probably a condition precedent to the authority of the commission to enter into contracts that require such consent and the

failure to obtain such consent would probably mean that the contract would be invalid and not enforceable. (see Collins v. Hydro Electric Commission of Renfrew (1948) O.R. 29.)

When a commission enters into a contract within its authority the contract will be binding on and enforceable by the municipality subject to the limitation that its performance will be excused if monies are not provided, in accordance with the Act and other requirements of the law, to carry out the contract. In order to avoid this possibility of non-performance of the contract, it is suggested that a party to a contract with a commission ensure, not only that the consent of the council has been formally obtained by a by-law where it is required under the Act, but that sufficient moneys will be available to carry out the contract. One way that this could be done is to make the municipality a party to the contract, as guarantor or otherwise, solely for the purpose of committing the council under s. 41(4) to provide sufficient moneys for its performance. It is suggested that this procedure would not be an exercise by a council of the powers, rights, authorities and privileges that should be exercised exclusively by a commission under s. 41(1).

The general principal governing the relation of the council of a municipality to a commission under s.41(1) of the Act, namely, that the council cannot directly control the exercise by a commission of its powers except where the consent of the council is required, does not apply to the establishment of a fluoridation system under The Fluoridation Act, 1960-61. Since a commission is a "local board" of a local municipality, the effect of s. 2(1) of The Fluoridation Act is that where a local municipality owns the waterworks that are operated by a commission, the council of the municipality may by by-law require the commission to establish, maintain and operate a fluoridation system in connection with the waterworks system. Under s. 3(1) where a fluoridation system has been established the council may by by-law require the commission to discontinue the system.

In Re City of Windsor & Windsor Utilities Commission decided in June 1962 the Court granted a mandamus order to the City of Windsor directing the Windsor Utilities Commission, in accordance with The Fluoridation Act, 1960-61, and the City by-law passed thereunder, to establish a fluoridation system in connection with the waterworks system of the City of Windsor and thereafter to maintain and operate the system.

For a more detailed analysis of the nature of the powers relating to waterworks that a public utilities commission can exercise in a municipality and the nature of the legal relation between a commission and the municipality and its council from the standpoints of the control that the council

can exercise over the activities of the commission and the extent to which the commission can exercise its powers free from such control I refer you to an article of mine entitled "The Law in Ontario Relating to a Waterworks Public Utilities Commission" in the July 1962 issue of the Municipal World magazine. This article is substantially the same as one published in the Canadian Bar Journal in June of 1962 and also appeared in the July 1962 issue of the Waterworks Information Exchange, Canadian Section, AWWA.

It would appear from s. 12 of the Act and sections 9 and 242(1) of The Municipal Act that a by-law of the council of a municipality is required in order to authorize the treatment of the water supplied by the municipality. The validity of the by-law would be tested by the standards set out in s. 12 of the Act, namely, that its purpose must be "to secure to the inhabitants a continued and abundant supply of pure and wholesome water.....". If a court found that the purpose of the by-law was not to produce a pure and wholesome supply of water the by-law would be invalid. This statutory limitation on the power of a municipal corporation to treat its water supply is illustrated by Metro Toronto vs. The Corporation of the Village of Forest Hill (1957) S.C.R. 569. The majority of the Supreme Court of Canada applied s. 43 of The Municipality of Metropolitan Toronto Act to a by-law providing for the fluoridation of the municipal water supply and concluded that the purpose of the by-law was not to furnish water for its accepted purposes only but to use the water as a means of rendering teeth less susceptible to decay, that is, as a means of effecting compulsory preventative medication and therefore held that the by-law was not within the standard of validity set out in s. 43 and was invalid. Since s. 43 of The Municipality of Metropolitan Toronto Act is in substance the same as s. 12 of The Public Utilities Act in regard to a standard of validity for by-laws regulating the water supply it is clear that under s. 12 of The Public Utilities Act the by-law in order to be valid must be for the purpose of rendering the water supply suitable or more suitable for the accepted purposes for which water is used and if the purpose of the by-law is the promotion of health the by-law will be invalid. Thus if a by-law required the treatment of water by means of water soluble vitamins or minerals for the promotion of health it would be invalid as being beyond the authority of the municipal corporation under s. 12.

A municipality or a commission supplies water either under a contract or in discharge of a duty to do so imposed by a statute. Generally speaking, the supply of water beyond the limits of the municipality is under a contract, although there are cases where it is pursuant to a statutory duty, and the

supply of water within the municipality is in discharge of a statutory duty, although in some cases it is under a contract.

Section 11(1) of The Act authorizes a municipality to supply water to owners or occupants of land beyond the limits of the municipality and s.s. 2 authorizes a municipality to enter into a contract for a term not exceeding 20 years for the supply of water within or beyond the limits of the municipality and to renew any such contract. Where a commission is operating the waterworks it would have the exclusive right under s.41(1) to exercise these powers to enter into contracts free from the control of the council provided it complies with all requirements of the law. One of these requirements is obtaining the consent of the council which owns the works furnishing the supply to the supply of water to or in any other municipality (s.41(5)(a)). If the consent of the council is not obtained, any contract made by the commission for the supply of water to or in any other municipality would probably be invalid. (Collins v. Hydro Electric Commission of Renfrew (1948) O.R. 29).

There is a question whether s. 11 necessarily creates legal capacity in a municipality or a commission to enter into a contract to obtain a supply of water from another municipality or a commission. However in view of the specific provision in s. 287(1) of The Municipal Act which empowers a municipal corporation, with the assent of the electors, to enter into a contract for the supply of water to it, it can be argued that s.11 should not be interpreted to create such capacity in either the municipal corporation or a commission and thereby avoid the requirement of the assent of the electors, set out in s. 287(1) of The Municipal Act. The result might be that although a commission, with the consent of the council, can enter into a contract under s. 11(2) to supply water to another commission or municipality, neither s.11 nor any other section of the Act empowers the other commission or municipality to enter into the same contract to obtain the supply. It would be necessary for the municipality that is to be supplied, itself to enter into the contract, pursuant to s. 287(1) of The Municipal Act with the assent of the electors.

The statutory duty to supply water is found in sections 9 and 55 of the Act. Section 9(1) imposes a duty to supply water to all public institutions in a municipality or within 3 miles thereof and belonging to or maintained by the Province. Unlike s. 55 there is no requirement that the public institutions be situated along the line of any supply pipe or that there be a sufficient supply of water. Under s.s.2 contravention of s.s. 1 carries with it a penalty of not more than \$500. Although s. 41(1) does not impose upon a commission the "duties" imposed upon a municipality by the Act, and the

Court in Re Public Utilities Commission of Thorold & The Town of Thorold (1927) 60 O.L.R. 429 held that the Act did not impose upon commission the duties and obligations with reference to providing waterworks that were imposed on a municipality by The Public Health Act it is clear that the duties found in sections 9 and 55 apply to a commission as well as to a municipal corporation (Holmberg v. Public Utilities Commission of Sault Ste. Marie (1966) 58 D.L.R. 2d 125; see also s. 49 of the Act).

Under s. 55 the duty to supply water would exist notwithstanding that the building to be supplied is situated outside the municipality which is supplying the water. In St. Lawrence Rendering Co. v. Cornwall (1951) O.R. 669 it was held that the City of Cornwall is subject to the duty imposed by s. 55 to supply water to the buildings of the plaintiff company that were situated along the line of the supply pipe installed for the purpose of supplying the company notwithstanding that these buildings were situated in the Township of Cornwall. At p. 684 Mr. Justice Spensst stated "Surely then, the words 'within the municipality' in s. 55, to have any meaning must mean not the supplier of the utility but the area in which the utility is supplied." In other words it would appear that the duty extends to the area of any municipality in which a supply line has been constructed from a different municipality which itself or by means of its commission is supplying water through the line.

In the Holmberg case the Court of Appeal held that the duty imposed by s. 55 is a broad one and that where a subdivider, under an agreement with the commission, had installed the watermain and appurtenances to service the subdivision, the water to be supplied by the commission which controlled the mains, the commission could not refuse to supply water to a homeowner in the subdivision merely because the subdivider had not tested the main as provided for in the subdivision agreement. It was also immaterial that under a commission by-law an agreement was required with the person to whom the water service was being provided before service would be made available and that no such agreement had been entered into. The commission's by-laws would not be construed to defeat the duty imposed by s. 55 nor could the duty be controlled by subdivision agreements.

The Court also held that a duty arose under s. 55 to provide electric power to the building although no secondary line or transformers, which would be necessary to feed power from the primary line to the houses in the subdivision, had been installed. The reason given was that the primary line was "any supply wire" within the meaning of those words in s. 55 and that the application of this section would not be defeated by the subdivider's default in installing the necessary secondary line and transformers.

At p. 133 the Court stated "If a public utilities commission has exacted a commitment from a subdivider to provide these facilities I do not agree that it can take advantage of his default to defeat a householder's claim for service unless there is no supply line at all from which it can be provided. Here there is one; and the fact that another is needed together with a transformer to make the service effective is not, in my opinion, a prerequisite to the statutory duty."

Since s. 55 applies equally to water or electric power the Court's decision could be interpreted to mean that if a building lies along the line of a transmission main the duty under s. 55 would arise notwithstanding that a distribution main had not been constructed to service the property. Such an interpretation could mean that a commission or a municipality would be under a statutory obligation to construct distribution systems to service buildings which are located along the line of a transmission main without regard to whether the cost of such construction is justifiable or whether the design for the waterworks system included the construction of distribution mains in the particular location. I think this decision would be limited to the situation where a municipality or commission had obtained an obligation from a subdivider, or otherwise, to provide the necessary distribution mains to service the particular area and the design of the waterworks system included distribution to this area. In other words it is suggested that the duty under s. 55 arises if a building lies along the line of a transmission main notwithstanding that there is no distribution main to supply it if there is an obligation, whether in a subdivision agreement or otherwise, to construct a distribution main to service the particular property. Support for this interpretation is found at p. 127 where the Court stated "The two agreements (subdivision agreements) are important, not because the applicants can found any contractual claims under them - clearly, they cannot - but because they have a bearing on whether the work done under them (if it was completed) brings the statutory duty under s.55 into play."

In the Holmberg case the Court noted that the watermain that had not been tested and charged with water was under the ownership or control of the commission. Section 55 should be interpreted so that the duty under it would arise only if the corporation supplying the water also owned or controlled the supply line; otherwise it might not be in a position to discharge the duty under s. 55 if the owner or person in control of the supply line did not permit the supply of water through it.

Section 55 is predicated on there being a sufficient supply of water but there is no indication in the section of the purposes or areas of the municipality for which the supply

should be sufficient before the statutory duty would arise. For example, if there is a sufficient supply of water for buildings situate along the line of the supply pipe outside the municipality but there is not sufficient for the needs of the municipality itself, would the duty to supply water arise? Does "sufficient supply" include the maintenance of a reasonable reserve for future growth? It is suggested that "sufficient supply" in the section should be interpreted to mean sufficient for all the purposes and areas which the supply line was designed to serve, including the supply of buildings along its line outside the municipality and including an adequate supply for future growth of such areas.

Sections 9 and 55 do not specify the quality of water that must be supplied under the statutory duties. In the St. Lawrence Rendering case, supra, the Court held that where water is supplied pursuant to a duty imposed by statute it is not and can not be supplied pursuant to a contract. It follows that there can be no standard of quality established by contract for water supplied pursuant to sections 9 and 55. It is suggested that the quality of such water would be the same as the quality that is required at common law of a supplier of water as set out by the trial Judge in the Munshaw case, infra, where he held that the water must be wholesome and ordinarily pure, i.e., it must be fit for ordinary human consumption and domestic use. It is also suggested that the Drinking Water Objectives 1967 formulated by the Ontario Water Resources Commission furnish a reasonable and acceptable definition of this standard of quality. A contrary view is implied in the Scottish, Ontario, Manitoba Land Co. case, infra. Here the statutes pursuant to which the water was supplied, did not specify expressly a standard of quality. The water that was supplied contained sand that damaged the plaintiff's hydraulic elevators. One member of the Court held that a duty was imposed by the statutes to supply water and went on to state "It is unnecessary to express any opinion as to the rights of the parties aggrieved or fancying themselves aggrieved by any breach of a statutory duty created under these Acts, as no such breach was proved by refusing to furnish water as required whatever may have been the quality of water supplied." This view appears to mean that so long as water is furnished as required there is compliance with the duty to supply water under the statutes even though the water may be of a quality unsuitable for ordinary human consumption and ordinary domestic use. In view of the responsibility of the Ontario Water Resources Commission under The OWRC Act for the control and supervision of standards of quality for public water supplies, which I will refer to later, it is suggested that the above quotation from the Scottish, Ontario, Manitoba Land Co. case does not express the law in Ontario at the present time.

For the protection of the public the Ontario Water Resources Commission exercises a supervisory function over waterworks. Thus under s. 30 of The OWRC Act, with certain limited exceptions, when a municipality, commission or other person proposes to establish, extend or alter waterworks the approval of the Commission must be first obtained to both the source of supply and the proposed works. Plans, specifications and an engineer's report of the water supply and the works to be undertaken must be submitted to the Commission. It is an offence to undertake or proceed with the works or to pass a by-law to raise money to finance the works without first obtaining the Commission's approval. The Commission is given certain powers to direct investigations and changes in the source of supply and the works where a waterworks is undertaken by any person without prior approval. The approval can be refused in the "public interest" or it can be granted on terms and conditions. A duty is imposed on any municipality or other person owning waterworks to provide the Commission with any required information and all works must be "maintained, kept in repair and operated in such manner and with such facilities as may be directed from time to time by the Commission." Some contraventions of this section are not an offence if carried out by municipalities or commissions because it is believed that s. 38 confers sufficient authority on the Commission in regard to municipalities to prevent such contraventions.

Section 38 imposes a duty on a municipality forthwith to "do every act or thing in its power to implement" a report of the Commission that the Commission is of the opinion that it is necessary in the public interest that waterworks be established, maintained, operated, improved, extended, enlarged, altered, repaired or replaced by the municipality. Failure to carry out this duty is an offence and the Commission, with the approval of the Ontario Municipal Board may itself implement the report and recover the expense of doing so by legal action against the municipality. The power of a municipality to implement the report implies that it has financial ability to do so and must comply with all requirements of the law. Thus where applicable, the duty to implement would require that the municipality obtain the approval of the OMB to the required expenditures (see s. 64 of The Ontario Municipal Board Act which applies "notwithstanding the provisions of any general or special act.....") and the approval of the Commission under s. 30. For example in Clarey v. Ottawa (1914) 5 O.W.N. 673, aff'd 6 O.W.N. 16 the effect of a report of the former Provincial Board of Health to the City of Ottawa under similar legislation was in question. The Court held that before the duty to implement the report arose and the municipality could pass valid by-laws for such implementation the approval of the Provincial Board to the

proposed works under a section similar to section 30 of The OWRC Act must be obtained and the detailed information about the works referred to in that section must be sent to the Board. In Dilworth v. Bala (1952) O.R. 703, aff'd 1953 O.R. 787, aff'd on other grounds 1955 S.C.R. 284, a report of the Department of Health to the Town of Bala under a section of The Public Health Act similar to s. 38 ".....that a waterworks system be installed in the said municipality....." was in question. The grounds for attacking the report were that the works as constructed differed in many respects from the plans of the works which had been approved by the Department and therefore that the works were not constructed to implement the report of the Department and also that the report was too vague to be implemented. Another ground of attack was that the Town had not passed by-laws authorizing and providing for the various acts for the initiation of the scheme and the installation of the system and for entering into the construction contract.

The Court held that the report need not be on a specific waterworks and held that "It is sufficient if it reports that a 'system should be established' and then subsequently approves the system which is planned." (At p.708 of the report of the trial). Plans and specifications for the works were approved by the Department and the approval of the Ontario Municipal Board was obtained before the municipality passed by-laws to implement the report. The Court also held that the variations in the works were not substantial, did not alter the character of the scheme and were not unreasonable and therefore the work was being done to implement the report, and that by-laws were unnecessary in the circumstances of the case. The Court based the decision on the lack of necessity for by-laws, in part, on the fact that the Town was under a statutory duty to construct the works to implement the report of the Department of Health.

It is interesting to observe the changing attitudes of the Courts to the desirability of enforcing reports for the construction of waterworks. Thus in the Clarey case at p. 674 of the report in 5 O.W.N. 673 the Court, in referring to the power in the Board to compel a community to assume a financial burden for the building of a waterworks system said "But being an exceptional and drastic power, it is obviously imperative that the conditions of its exercise must unquestionably exist and be scrupulously observed." The judgments in the Bala case indicate a liberal interpretation of the conditions for the exercise of the statutory power designed to make the report effective rather than to invalidate it on technical grounds. In Re PUC of Thorold & The Town of Thorold (1927) 60 O.L.R. 429, the Court held that no duty was imposed on a public utilities

commission to implement the report of the Provincial Board of Health under The Public Health Act because the duties of a commission were only those set out under The Public Utilities Act. Under The OWRC Act "municipality" is defined to include a public utilities commission and it may be that notwithstanding the wording of s. 38 indicating that a report is to be made to a municipal corporation and the decision in the Thorold case, s. 38 would be interpreted as being applicable to a public utilities commission as well as to a municipal corporation. Certainly where a commission has control of the waterworks the purpose of the section might be frustrated if a report could only be made under it to a municipal corporation because of the separation of authority between a commission and a corporation referred to earlier.

A contract for the sale of water may require expressly that water of a specified quality be supplied and in such event failure to do so is a breach of contract and may result in liability for damages.

Where a contract does not expressly require a specified quality of water to be supplied there may still be liability for the supply of an unsuitable quality of water based on breach of a warranty of fitness that a Court may imply in the contract. In Read v. Croydon Corp. (1938) 4 A.E.R. 631, there is a suggestion by the Court that in certain circumstances the warranty that the water supplied will be reasonably fit for the purpose for which it is required will be implied in a contract for the supply of water whether it be regarded as one for the sale of goods and/or for the sale of services. Assuming water comes within the definition of "goods" in The Sale of Goods Act R.S.O. 1960 c. 358, s. 15 and a contract for its supply is subject to this Act, a warranty of fitness will be implied where the buyer, expressly or by implication, makes known to the supplier the particular purpose for which the water is required so as to show that the buyer relies on the supplier's skill and judgment for its quality, and it is in the course of the supplier's business to sell such water. If the water is not reasonably fit for the purpose for which it is required and the buyer suffers damage as a result the supplier may be liable for a breach of the implied warranty of fitness whether or not he was negligent.

Before a warranty of fitness can be implied it must be established that the municipality or commission entered into a contract for the sale of water. The St. Lawrence Rendering case clearly holds that where water is supplied under a duty imposed by statute (while the duty under s. 55 of The Public Utilities Act was involved in this case, such a duty would also arise under s. 9 of The Public Utilities Act or under a report or

order made under s. 16(1)(a), 38(1) or 46a of The OWRC Act) there can be no contract for the sale of the water and therefore there can be no implied warranty that it is reasonably fit for the purpose for which it is supplied. (See also Scottish, Ontario & Manitoba Land Co. v. City of Toronto (1899) 26 O.A.R. 345 at p. 350 and Read v. Croydon Corp. (1938) 4 A.E.R. 631 at p. 647). The dictum of the Court in the St. Lawrence Rendering case that there is a common law duty to supply water, apart from statute, is not only inconsistent with the contrary implication that may be drawn from the Seguin case supra, but has been disapproved by the Court of Appeal in the Holmberg case supra. Once it is found that water is supplied under a contract of sale it must also be shown that the buyer made known to the seller the purpose for which the water was required so as to show that he relied on the seller's skill and judgment for its quality and that he suffered damage as a result of such reliance. Damages cannot be recovered for breach of an implied warranty if damage is caused by the consumer's own carelessness in continuing to use water that is not reasonably fit for his purpose after he could have reasonably discovered that the water was unfit, have ceased to use it and thus prevented the damage. Thus in the Scottish, Ontario & Manitoba Land Co. case at p. 348 Chief Justice Burton stated that there was no implied warranty even if the Court had found a contract for the supply of water because, "It surely cannot be said that there was any warranty that the water was fit for the purpose for which it was required. Here was the sale of an existing article upon which the purchaser had the opportunity of exercising his own judgment and there is no fault on the part of the sellers and even though the defect might not have been discoverable until after the water was in use, whenever it was ascertained it was open to the plaintiff to cease to take it and prevent the damage which subsequently occurred." In other words, not only did the plaintiff not rely on the defendant municipality's skill and judgment in supplying the water but, in law, the damage was not caused by the sand in the water but was caused by the plaintiff's carelessness in continuing to use unfit water after the plaintiff could have discovered this fact and have ceased to use it. Other examples might be the continued use of water discoloured by iron for laundry purposes or the consumption of water with an offensive taste after the consumer could have discovered this, ceased to use the water and prevented the damage.

The converse situation is illustrated by the Croydon Corp. case where the action was based on the plaintiff contracting typhoid fever from drinking impure water supplied from the defendant's wells. In such a case the plaintiff clearly relied on the defendant corporation's skill and judgment in supplying the water because the plaintiff could not have, by any reasonable

examination of the water, have determined that it was unfit for human consumption and the plaintiff suffered damage as a result of such reliance. The Court stated that if the water had been supplied under a contract of sale, whether the contract be regarded as one for the sale of goods and/or the sale of services, a warranty of fitness would have been implied in the circumstances of the case. These circumstances appear to be that where water is supplied for human consumption and ordinary domestic use, the supplier knows and accepts that the public (by its use of the water without a reasonable opportunity of testing its safety) relies on his skill and judgment for its fitness.

In the absence of any applicable standard of quality prescribed by statute, law, or regulation, or by contract so long as the purpose for which the water is required is ordinary human consumption or ordinary domestic use the water will be reasonably fit therefor and there will be no liability based on implied warranty if the water meets generally accepted standards of quality for such a purpose. In 1959 "The Public Health Service Drinking Water Standards 1946" of the United States Public Health Service were accepted by the trial Court in Munshaw Colour Service v. Vancouver (1960) 22 D.L.R. 2d 197 at p.206 (reversed on other grounds by the Court of Appeal in 1961 29 D.L.R. 2d 240 and by the Supreme Court of Canada in 1962 33 D.L.R. 2d 719) as a generally accepted guide in Canada and the United States as to the quality of water that would be fit for human consumption or ordinary domestic use. These Drinking Water Standards were revised by the United States Public Health Service in 1962. In Ontario there is no standard of quality prescribed by statute for the water supplied by a municipality or a public utilities commission. Section 47(1)(g) of The OWRC Act provides that, subject to the approval of the Lieutenant Governor-in-Council, the Ontario Water Resources Commission may make regulations "prescribing standards of quality for potable and other water supplies", but no such regulations have yet been made by the Commission. Section 16(1) of the Act provides that it is the function of the Commission and it has power "(a) to control and regulate treatment of water for public purposes and to make orders with respect thereto." This power relates to specific instances of water treatment and does not relate to a general standard of quality for water used for public purposes. The Commission has made several orders under this section relating to water treatment in municipalities.

The Commission has formulated Drinking Water Objectives 1967 which differ from the United States Public Health Service Standards 1962 only in respect of colour and turbidity requirements. It is suggested that in Ontario a Court should accept the Commission's Drinking Water Objectives 1967 as a definition

of the standard of quality required for water to be fit for ordinary human consumption or ordinary domestic use rather than the United States Standards in any case where the difference between the Objectives and the Standards are of importance because of the responsibility of the Commission under The OWRC Act for the control and supervision of standards of quality for public water supplies.

The supply of water that meets the United States Public Health Service Standards 1962 or the Commission's Drinking Water Objectives 1967 would not be a defence to liability based on breach of contract if a contract term expressly required water of a better quality, or to liability based on breach of implied warranty of fitness if the consumer made known to the supplier that water of a better quality was needed for his particular purpose and the circumstances show that when the contract of sale was made, the consumer relied on the supplier's skill and judgment to furnish the better quality water, and that in law, damage was caused to the consumer by such reliance.

The quality of water to be supplied under a contract of sale should be determined by the express agreement of the parties and should not be left to be determined by a Court that may or may not imply a warranty of fitness. Thus in all cases of a contract for the sale of water it is advisable to deal expressly with the quality that must be supplied and to negative expressly the existence of an implied warranty of fitness. The inclusion in a contract of an express warranty as to quality does not itself negative a warranty of fitness that may be implied under The Sale of Goods Act unless the express warranty is inconsistent with the implied warranty.

Another legal basis of liability at common law for supplying unsuitable water is the negligence of the supplier. Generally speaking this may be defined as doing that which a reasonable man in the position of the supplier would not do or omitting to do that which a reasonable man in the position of the supplier would do, thereby causing harm to those users whom the supplier should have reasonably foreseen would be harmed. In other words while a reasonable mistake or omission in the treatment or supply of water which causes damage to such users does not result in liability for negligence, an unreasonable mistake or omission which causes such damage might result in such liability.

The standard of reasonable care required is flexible, varying with the particular circumstances of each set of facts. It is not based on "hindsight", but on reasonable foresight at the time the alleged unreasonable act or omission took place. The following cases illustrate liability based on negligence.

In O'Dell and Mitchell v. City of London (1919) 17 O.W.N. 284, the plaintiff recovered damages for loss of goods and interference with his business caused by water escaping from a municipal water main because of the City's negligence in the following respects:

- "1. The maintenance of the hydrant attached to the sidewalk in such a manner as to prevent the frost jacket attached thereto from performing its function properly.
2. The failure of the defendants to maintain the hydrant in question free of water and ice whereby the hydrant was prevented from operating in its proper manner.
3. Unreasonable delay in shutting off the water after the break in the main by reason of the defendant's failure to maintain a proper system of men and appliances to attend promptly to breaks in cold weather, and proper means, for example charts, for the purpose of enabling repair gangs without undue delay, to locate the valve or valves to be shut off."

The Court rejected the City's defense that the main break was caused by an Act of God for which the City was not responsible, on the ground that this defense is available only if it had been shown that the City had taken all reasonable precautions and that the damages complained of not only might, but must have happened independently of its neglect.

In the case of Canadian Pacific Railway Company v. The City of Toronto (1954) O.R. 535, a cast iron water main laid in 1912 by the defendant City burst in 1950, causing extensive flooding of the plaintiff's premises with damage to its property and equipment and to goods in its custody. The plaintiff claimed damages both on the ground that Coxon, an employee of the defendant had, through incompetence, failed to turn off the water as quickly as it should have been turned off. The break occurred at 3.40 a.m. and the water was shut off at 6.10 a.m. after about 7 million gallons had escaped. There was no delay in reporting the break or in getting an emergency crew to the scene. Coxon, in charge of the crew, mistakenly assumed that the break had been in a 6" main rather than in a 36" main and, after the crew had worked for about 1-1/2 hours trying to locate the break, sent for his supervisor, Atcheson, who quickly located the break and turned off the water. The Court held that there was no negligence. The pipe had been laid in accordance with sound engineering practice in 1912 and the fact that subsequent research, the results of which were first published 30 years later, had disclosed that cast iron was not as suitable for large pipes as had been thought, did not mean the defendant was negligent in failing to take up and replace about 35 miles of cast iron water mains.

With respect to shutting the water off, Mr. Justice Judson at p. 541 said, "It would be very easy for me sitting with the photographs in front of me to say that Coxon should have suspected at once that the break was in the 36" main and not in the 6" domestic main. But the problem that faced Coxon was a difficult one. There was a man with 22 years experience in the Water Works Department and an excellent record. He had been on permanent service for the emergency department for six months before this break occurred and for some years before that he had been doing relief work with the emergency service. He was qualified by training and experience to do the job to which he was assigned. A man who goes to deal with a break of this kind has to size up the situation and then act according to his judgment. I am satisfied that Coxon put forward his best efforts and used his best judgment. I am equally satisfied that his best judgment was not equal to that which would have been used by Atcheson if he had been on the job from the beginning. But Atcheson's knowledge of the system was something out of the ordinary. It would be unreasonable to expect the Water Works Department to have on duty at all times a man with the qualifications of an engineer or the ability of this particular superintendent. I think Coxon was a reasonably competent man on the job to which he was assigned and that he did the job to the best of his ability and with diligence. Both Mr. Fitzgerald and Mr. Storey say that 1-1/2 hours would be a reasonable time within which to deal with a situation of this kind. It may be that Coxon should have summoned assistance before he did, but I cannot find the City employed an incompetent servant or that he did his job incompetently or negligently."

In an English case Read v. Croydon Corporation, supra, the defendant corporation owned and operated two wells from which it supplied water to the surrounding borough. The plaintiff claimed damages as a result of contracting typhoid fever from drinking water supplied from the wells. The disease was introduced into the water either from the gathering ground or by a carrier of the disease described as "Case A", who was working in the wells during repairs. The Court absolved the defendant from negligence in selection of the gathering ground and of the workmen, but held there was negligence in omitting precautions in the form of continual analysis of the water and a systematic policy of chlorination and filtration, during the repair of one well. At p. 646 Mr. Justice Stable states "In my judgment the Corporation..... failed in its duty in two major respects. From July 1936 when the chlorinating plant first came into use, the extent to which this precaution was enforced was haphazard, ill-considered and inadequate. The decision at the moment of greatest peril, when the water was being put into supply while the men were working on the well, to omit the customary precautions of filtration

and chlorination, looked at, not in the light of what we know now, but in the light of what was known then, was in my judgment inexcusable Throughout the whole of this case no evidence was adduced to indicate that any one member of that Committee made any enquiry or took any step whatever to formulate any established policy in connection with chlorination or to set up machinery to put that policy into effect or to provide a proper system of supervision and inspection to insure that any policy that was adopted was being efficiently, regularly, systematically carried out While the probability looked at from the point of time immediately preceding these happenings that the water could be infected by Case A, was utterly remote, the bacteriologist or water engineer responsible for the supply of water to an urban community, if he discharges his duties properly, must be constantly on his guard against innumerable risks. Though the chance of any one of the risks materializing into a peril is remote, collectively and in the aggregate the danger is real and substantial." At p. 643 he states "no log book was kept showing whether or not chlorination was enforced at any particular time or when it was enforced or the dosage that was adopted. It is obvious that the practice of chlorinating the water or of increasing the dose after and in consequence of an unsatisfactory analysis, was not satisfactory in as much as that chlorination was thus used as a remedy for an existing evil rather than as a preventive of a potential danger which the experience of years had proved to have threatened."

In the Munshaw case supra, the plaintiff carried on the business of photo finishing and film developing and used a large volume of water in its automatic processing machines which it purchased from the defendant, the City of Vancouver. Films were damaged in the wash tanks by sediment in water which contained quantities of silt and a very high content of iron oxide. The plaintiff had a filter system which was ordinarily effective to eliminate from the water supplied particles of sediment that would damage films but in this case the filter system broke down and permitted larger particles to enter the tanks. On a particular day when the films were damaged the City had been carrying on sewer clearing operations by a process of "dragging" and "flushing" the sewer pipes with water from a hydrant located about 1250' from the plaintiff's premises. It was established in evidence that the use of water from hydrants for this purpose was a common practice in Vancouver and other large cities in Canada and the United States and no warning of the sewer clearing operations was given by the City to the plaintiff.

In reversing the trial Judge the Court of Appeal and the Supreme Court of Canada held that the City was not negligent in opening the hydrant for sewer flushing purposes because it was following ordinary routine practice which could not involve

it in liability to a consumer of water which had to be of a quality higher than that in ordinary use. While it may have been foreseeable that sediment would be stirred up in the process of flushing the sewer there was nothing to suggest that it would be an unusual amount of sediment and the turbid water would cause no harm to the ordinary consumer. The use of the procedure of flushing sewers by hydrants was an ordinary every day procedure and in itself could not be negligence. There was no evidence to suggest that the City should have known that the use of the hydrant would stir up more silt than it had done on other occasions. There was no reason for the City to warn anyone of a simple operation of this kind. To impose on the City the duty of notice in circumstances such as these was to require a standard of perfection, the circumstances being that the consumer was 1250' away from the particular hydrant, that no problems had arisen on the previous two days when the sewer had been flushed and that hydrants had to be opened frequently for many reasons and in this case the hydrant had been properly opened. At p. 726 of the report in 1962, 33 D.L.R. 2d 719, the judgment of the majority of the Supreme Court stated "Even if it were foreseeable that the use of the hydrant would result in the delivery to some consumers of turbid water this in itself amounts to nothing. Everybody is familiar with turbid water and knows what to do with it. Although the waterworks officials had they thought of the matter might have concluded that there must be people in the City engaged in the processing of film, the City should not have to pay damages because a routine operation results in delivery of turbid water to a film processing plant. I have not the slightest doubt that the City employee, in going about his work never thought about the effect of opening the hydrant upon water consumers of peculiar sensitivity. In my opinion it is not negligence to fail to turn one's mind to this problem. It would be impossible to do anything with a waterworks system if the City had to consider these minutiae, in relation to routine operations. Those who have particular requirements, and in this case it was a particular requirement over and above water of ordinary standards, must deal with the problem as part of their ordinary operating procedure

It was not negligence to fail to clean out the sediment and rust in the pipes. It was not shown that Vancouver had more of this than any other municipality But the presence of sediment and rust in cast iron pipes is an every day matter in the operation of a waterworks system and to say that the City must periodically flush the pipes to remove the sediment and rust so that this particular kind of consumer will not be affected is again to impose too high a duty on a municipal waterworks system."

The statement of the law by the trial Judge in the Munshaw case as to the quality of water that must be supplied at common law was not affected by the reversal of the decision in the Court of Appeal and the Supreme Court of Canada. In fact I think these judgments impliedly accept his judgment on this point. His statement was that at common law the quality must be "wholesome and ordinarily pure" that is, fit for ordinary human consumption and ordinary domestic purposes. The quality need not be chemically pure nor suitable for the particular purpose of any consumer whose operations may require a better quality. However it is important to note that there may be liability based on negligence to a consumer who needs water of a particular quality notwithstanding that the quality of the water that is supplied to him is fit for ordinary human consumption. An example is Goldfish Supply Co. Ltd. v. The Village of Stouffville decided by the Court of Appeal of Ontario in September 1966. It is to be regretted that no reasons were given for the decision and that it was not reported. A large number of goldfish maintained by the plaintiff in fish tanks in its plant in the defendant municipality were killed after restoration of the water supply to a reservoir in the Village which had been previously emptied of water due to a power failure and the resulting stoppage of a pump supplying water to the reservoir. The trial Judge in his reasons for judgment found that "the precautions taken by the Town were not in accordance with regular standards and there was negligence." However he held there was no evidence that "the influx of chlorine.....made the water unfit for human consumption" and that the duty was "to supply water that reaches the consumer in a state fit for human consumption and I don't think that this duty extends beyond that; it wouldn't extend to a consumer of an extraordinary sensitivity who in my view would be obliged to take his own precautions." Accordingly he dismissed the action against the Village.

The Court of Appeal reversed this decision and held the Village liable. The argument of the solicitor for the successful appellant was that the only reasonable explanation for the plaintiff's loss was over-chlorination and that the plaintiff's fish had never been damaged by the Village water supply before or since the loss. Since I am advised that the Village had continuously chlorinated its water supply before and since the loss to the plaintiff it would appear that the Court of Appeal accepted the theory of the loss put forward by the appellant. I am advised that the only way over-chlorination could have taken place in this particular waterworks was that the concentrated chlorine solution was siphoned from the chlorine container into the Village feeder main because the pressure in the main dropped below the level of the chlorine solution in the container. It would appear from this decision of the Court of Appeal that a supplier of water who fails to take reasonable

precautions in the operation of the waterworks might be liable in negligence for damage caused in law, to the property or operations of a consumer even though water supplied was fit for ordinary human consumption. The Munshaw case can be distinguished because the Court of Appeal and Supreme Court of Canada found that there had been no negligence in the operation of the waterworks and also that the damage had not been caused by any action taken by the supplier of the water but had been caused, in law, by the breakdown of the consumer's own filtering equipment.

The duty of the waterworks operator is relative to the treatment and supply facilities with which he has been provided, to the conditions which he knows about or reasonably should know about in his locality, to the past experience in the locality with water pollution and to the advice received from public bodies like the Ontario Water Resources Commission and the Department of Health. To be held liable for negligence he need not know that the water supply falls below the required standard of quality. It is sufficient for liability to be established if a reasonable man in his position and knowing what he knows, would have known that the water is below the required standard and would have taken precautions which he did not take. For example, in many waterworks there are no facilities for chlorination or other chemical treatment of the water or for filtration. Raw water is pumped from a well into the distribution system. In small communities where there has been no previous history of polluted water, it may be reasonable to take and test only two samples a month of the raw water. But if the waterworks operator knows or reasonably should know of factors which might adversely affect the quality of the water it would be necessary for him to increase the number of samples for testing. Moreover he must carefully inspect any well to see if there are cracks or other construction defects that might allow substances into the water that might pollute it. If there are repairs to be made or if the well area has been flooded recently it may well be necessary for him to stop the supply of water from the well and manually chlorinate it before using it again. It may also be necessary to take these precautions for the distribution system and even to take steps to warn the inhabitants if untreated water has entered the distribution system. It is reasonable that in every community the waterworks operator should be familiar with emergency chlorination procedures and have the equipment available for carrying out such procedures if necessary. Wherever siphonage is possible in the particular waterworks it is reasonable for a spring loaded check valve to be installed on the discharge line from the pump feeding chemicals into the main in order to prevent it. Such an installation would have prevented the damage in the Goldfish Supply case. The waterworks' operator must be on constant alert to the ever present

and ever changing threats to the quality of the water supply. If he takes all reasonable precautions, which will vary with what he knows or should know about local conditions, he will not be liable for negligence even though the municipality or public utilities commission may be liable for not providing him with adequate treatment facilities if such facilities should have reasonably been provided.

In waterworks where chlorination is carried out the following are some of the duties which if not performed or not performed properly might cause water of unsuitable quality and resulting damage and liability for negligence. The operator must judge as to how often he should check the chlorine content of the water in order to determine if the proper chlorine residual is being continuously maintained. If the quality of the water is subject to frequent change, as in Port Colborne, the operator may need to check the chlorine residual by hourly sampling. If the quality of the water is quite stable, as in St. Catharines, daily sampling may be sufficient. He must interpret the results of his checks correctly and he must maintain the correct amount of chlorine residual in the water at all times by properly adjusting the quantities added to variations in the quality and quantity of the water supply. He must have a sufficient supply of chlorine on hand at all times and not only accurately add the chlorine to the water but make certain that the chlorine is being added continuously. He must insure that the chlorination equipment is working at all times and that it is connected properly so that for example, in the event of fire unchlorinated water will not escape into the water mains by means of the fire pump. He must also insure that sufficient chlorine is removed so that the water will not have an offensive taste. Where proximity to oil refineries, as in Port Credit, or coal deposits, as in Port Colborne, creates a risk of chemicals, such as phenol, entering the water either alone or in combination with a chlorine residual causing an offensive taste, it is necessary to systematically and frequently sample the water and to add the correct amount of a chemical such as chlorindioxide to eliminate the offensive taste. It may even be necessary temporarily to turn off the water supply and obtain an alternative supply.

With respect to filtration the standard of care required from the waterworks' operator is relative to the facilities with which he has been supplied. If he has been supplied with a properly designed filtration plant he will be responsible for supplying water which meets the turbidity standards of the Ontario Water Resources Commission Drinking Water Objectives 1967. If he has not been supplied with the proper filtration equipment he will not be responsible for meeting this standard but he will be responsible for maintaining the standard of turbidity to the full extent of the filtration

equipment available to him. Where properly designed filtration equipment is available the responsibility of the operator would also extend to checking the pH regularly so that there will be no probability of damage to service pipes from excess acidity due to the addition of conditioning chemicals. For example in Seiden v. Passaic Valley Water Commission (1938) 199 A. 420, a United States Court held a company public utility liable for damages based on negligence because the water it supplied contained sediment that damaged a consumer's plumbing. The Court held that the water supplied must be harmless to the common and ordinary means of delivery of it to a consumer. Not only is filtration important from the standpoint of taste and appearance but inadequate filtration and excessive turbidity of the water have a direct adverse effect on the bacteriological quality of the water and therefore on the maintenance of a proper chlorine residual.

Even if water is supplied that is of an unsuitable quality and damage results from the use of such water there will not be liability for negligence if the failure to supply water of a suitable quality is not caused by unreasonable acts or omissions on the part of the supplier or if it is caused by an act or event beyond the control of the supplier. For example if the unsuitable quality resulted from an unusually high tide, frost of exceptional severity or a situation where the supplier had no knowledge of or no reason to suspect the possibility of contamination there would likely be no liability based on negligence. In addition the supplier may be relieved of any liability where, having knowledge of an existing or possible danger, it took every reasonable step to warn users that the water was liable to be harmful by reason of contamination.

Where a consumer uses water with an offensive taste, a discoloured appearance or any other clearly observable deficiency that may be due to inadequate chemical treatment or filtration, it may be difficult to establish liability or to recover full compensation for liability based on negligence. The reason is that although such water may not meet the standard of quality required at common law it is possible to argue that any damage was caused or materially contributed to in law, by the contributory negligence of the consumer and not by the unsuitable quality of the water. Such an argument is possible if the consumer could have reasonably observed the deficiencies and have stopped the use of the water before any damage was caused. The law of negligence does not compensate for reasonably avoidable damage even though the water supplied does not meet the required standard of quality because of unreasonable acts or omissions in its treatment and supply.

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